ENGINEERING NEWS

A JOURNAL OF CIVIL, MECHANICAL, MINING AND ELECTRICAL ENGINEERING

Vol. 62. No. 2. 220 Broadway, New York, July 8, 1909.

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Erection of the New River Bridge at Glen Lyn, Va., by the Cantilever Method.

By L. L. JEWEL,* Assoc. M. Am. Soc. C. E.

The New River Bridge of the Virginian Railway, recently completed, is located at Glen Lyn in Giles County, Va., at Mile Post 331.3 west from Norfolk and about a half-mile east of the West Virginia state line. It is a single-track deck structure throughout, 2,155 ft. long over all, 112 ft. above mean low water and about 90 ft. above the Norfolk & Western Ry. tracks which pass underneath on the west shore of the river. The alinement is tangent for its entire length, with a uniform grade of 0.8% down to the east or in the direction of heavy traffic. The crossing is made at an angle of 55° 30′ with the stream, which is about 17 ft. deep for nearly its full width of some 500 ft. The current is fairly swift even at low water and is subject to flood stages of an additional 20 ft.

The superstructure, designed for Cooper's E-60 loading, weighs 2,230 tons, or practically one ton per lineal foot of bridge. There are five channel spans of 136 ft. riveted trusses, supported on four solid concrete piers in the river and a special steel tower on each bank. The towers are set square to the axis of the bridge, but the concrete piers are set to coincide with the direction of the current, requiring a skew of 8 ft. 3 ins. in the ends of the spans they support. A truss span of the same type, 125 ft. long and carried on steel towers at each end, crosses the Norfolk & Western tracks. Viaduct approaches of 60-ft. intermediate spans and 30-ft. tower spans, plate-girders, are used at each end of the bridge, being 1,140 ft. and 180 ft. long at the east and west ends respectively. The arrangement is clearly shown in the diagram Fig. 1.

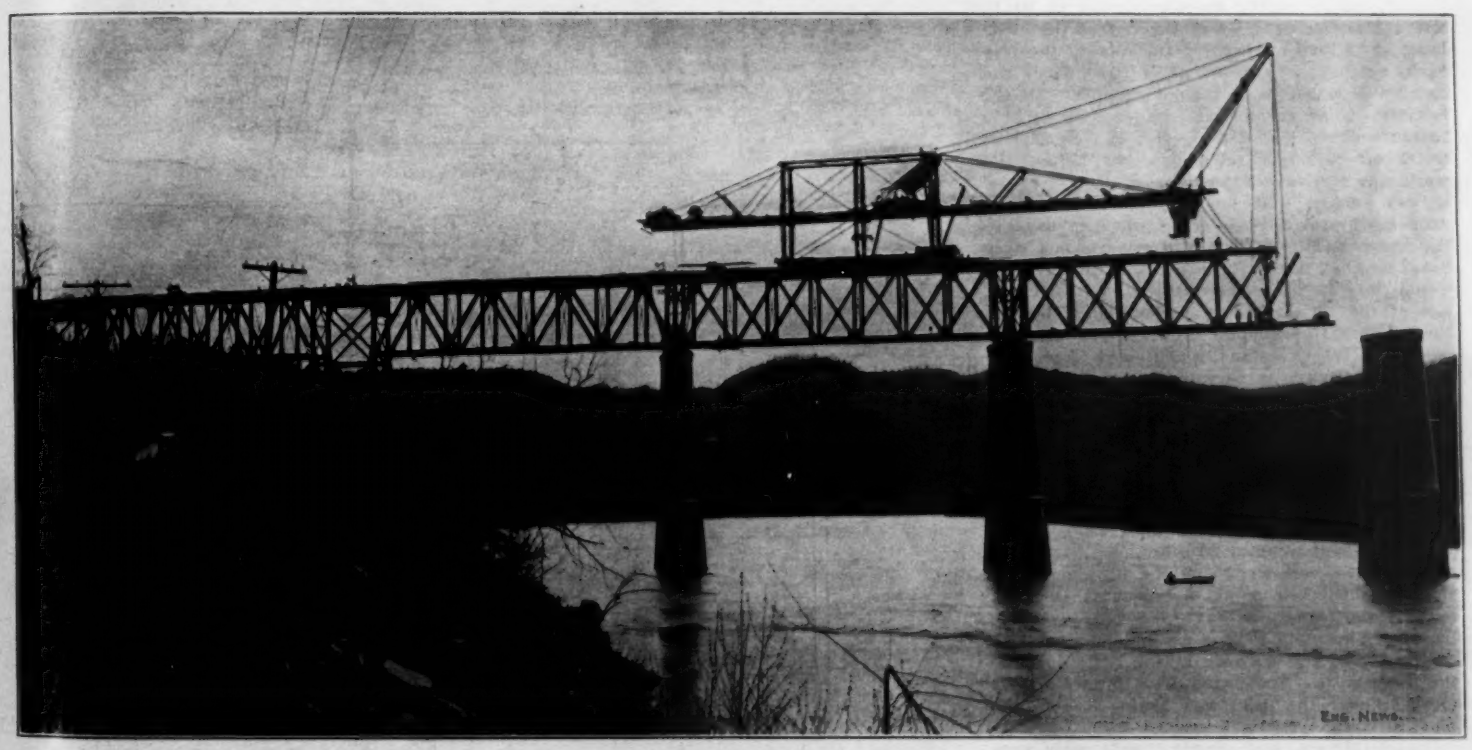
All trusses are riveted Warren trusses 26 ft. deep c. to c. of chords, and are spaced 12 ft. apart. No floor system is provided, the deck of extra heavy ties being carried directly on the top chords. The heaviest truss members are the center sections of the bottom chords, four panels long, which weigh 10½ tons. The 60-ft. girders of the approaches, weighing 13 tons each, are the heaviest single pieces that were handled.

All materials and erection equipment were delivered at the bridge site over the Norfolk & Western tracks. A storage yard was secured near the west end of the bridge (Fig. 3) about 90 ft. below grade, through which two parallel standard-gage tracks were laid, 18 ft. on center, and connected to the Norfolk & Western with a single switch. The material was all loaded and stored along these two tracks with a 15-ton locomotive crane of 40-ft. boom, which, working on one track, could unload a number of cars set in on the other track without a shift. Owing to other circumstances of erection all material for the bridge was delivered before any erection was started except that during intervals between shifts the unloading crew erected the first 60-ft. span and the two towers of the west approach with a gin-pole and hoisting engine. The approaches and all truss spans were stored systematically, the material first required being placed on top of the piles in each case.

To hoist the material to grade, an ordinary 50-ft. boom stiff-leg derrick of 15 tons capacity was mounted on a timber tower built adjacent and braced to the first steel tower of the approach. This one derrick, operated by a hoisting engine on the ground below, hoisted all material to grade as required, the traveler being served from this point by ordinary 15-ton hand trucks.

The erection was accomplished with a 15-ton electrically operated steel traveler with a 100-ft. overhanging jib, described in detail in Engineering News of Oct. 8, 1908. The power car for this traveler, and the usual compressed-air plant mounted in a separate car, completed the equipment for the work.

The 125-ft. truss span over the Norfolk & Western tracks was erected on falsework, shown by Fig. 2. A pair of 60-ft. girders from the east approach was supported on timber bents and braced together with timber cross-braces. The girders were set 12 ft. on centers, bringing



SETTING CLOSING SECTION OF BOTTOM CHORD, THIRD RIVER SPAN.

*Manager of Erection, McClintic-Marshall Construction Co., Pittsburg, Pa.

them directly under the plane of the trusses. A floor of the regular bridge ties was then laid on the girders and the trusses assembled on this. All bottom chord splices and the bottom connections of the main diagonals were fully riveted and the top connections filled with drift pins

TEMPORARY CONNECTIONS FOR FIRST SPAN.—The first span erected cantilever required some special arrangement to make it self-supporting, on account of the 30-ft. plate-girder tower-span between it and its anchorage, the 125-ft. span over the railroad tracks. The arrangement shown by Fig. 4 was employed therefor. For the tension chords a pair of top-chord middle sections, belonging to another of the river spans, was used, a splice being so arranged in the top chord of the span to be erected that the temporary chord would splice into it and also reach over the towers to engage the top chord of the anchorage span. This and the bottom-chord splice are shown somewhat more in detail by Fig 6. As the splice occurred in the first panel of the truss, and the end diagonal connects at the bottom, these ties could be removed after the erection was completed and the span landed, without interfering with the integrity of the trusses. The brackets for the support of the 30-ft. girder span were riveted to the end posts ½-in. below their proper position, an amount just equal to the thickness of the top cover-plate of the temporary chords. These chord sections had been shipped with the bottom lacing and tie-plates bolted in place, instead of riveted; these were removed in the field, so that the members were open on bottom full length, and the chords were then set down astride the permanent girders, bringing them into direct line for the splice and at the same time relieving them of any bending stresses due to the traveler wheels when working directly over the tower. On the completion of the span and the removal of the chords, ½-in. bearing plates were inserted under the ends of the girders to raise them to their proper elevation. In addition to the splice plates for these chord ties, special material was provided for the first panel of the top laterals (in this first span only; in all other spans the regular laterals could be put in place at once).

To carry the thrust of the bottom chords across the tower to the bottom-chord of the 125-ft. span, the end posts from the square end of the last (easterly) truss span were used as temporary bottom-chord members (see Figs. 4 and 6). As these posts were designed for the reaction of the adjacent 60-ft. girder span of the approach viaduct, as well as a half panel-load of the truss, they were amply heavy for the purpose. They were erected in line with the chords, with the cross-girder and permanent sway bracing (these are shown at C, Fig. 8) bolted in place to give them lateral stability. Plate-and-angle spacers were bolted between them and the permanent longitudinal strut at

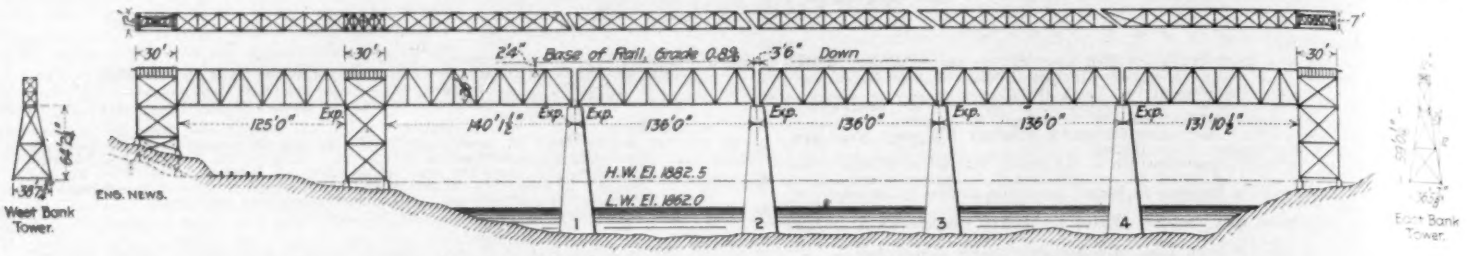


FIG. 1. OUTLINE DIAGRAM OF NEW RIVER BRIDGE NEAR GLEN LYN, VA., VIRGINIAN RY.

and bolts divided about two to one before the falsework was removed.

Cantilever Erection of River Spans.

All the other truss spans were erected as cantilevers for their full length, and were landed on their bearings before any floor deck or other superimposed load was put on them.

TRAVELER.—In order to do this it became necessary to increase the overhang of the traveler to about 150 ft. This was accomplished by mounting two standard 60-ft. 10-ton steel booms at the extreme end of the jib, as may be seen in the half-tone, Fig. 10. The traveler was originally designed for a load of 15 tons in addition to the weight of the crane bridge and trolley, which was 24 tons. The weight of the two booms and rigging was under 5 tons, which at the greatest radius of 50 ft. left a carrying capacity at the end of the overhang of 32 tons when the crane was all the way back against the tower, or 8 tons when the crane was out to its extreme position. As the heaviest truss member to be erected by the booms weighed less than 7 tons, and as all material erected was self-supporting before the work was carried beyond the reach of the crane, it was evident that the traveler could not be overloaded provided the crane was run back to the tower and kept there while the booms were handling their loads. The overload in any case would have been a comparatively small percentage of the jib's capacity, perhaps 10%, but all the men in the erection crew, particularly the cranemen, were continually cautioned to see that the crane was run back before picking up a member with the booms. The most severe load on the traveler was obtained when the steel bent supporting the east truss was erected. The material was lowered to the ground and assembled with the top away from the traveler and was then "tripped" up on its pedestals before the last two panels of the span were erected. The assembled bent weighed 20 tons.

The booms were seated on the jib about 8 ft. c. to c., with their points flared apart to 12 ft. so that the load lines hung vertically over the trusses. The topping lines ran all the way back to the top of the traveler tower, 100 ft. behind the boom-seats. The booms were braced rigidly together near the point and again close to the seat with steel struts borrowed from the east approach towers, and diagonal rods were provided to overcome the tendency to swing sidewise due to the eccentric pull of the topping lines. This transverse component was only about 4% of the stress in the topping lines, but owing to the uncertainty of its action and the length of the diagonals, 1-in. round rods were used. The booms were rigged with 1½-in. manila lines and were operated by the regular electric engine on the deck of the traveler.

The first six panels of each span were erected complete with the electric traveling crane, while the booms were used to place the shoes, end chord-sections and the last five web-members of each truss, these members being carefully carried out to the end of the finished work and landed on the chords there by the traveling crane ready to be picked up by the booms. The diagrams Figs. 4 and 5 show the order of setting of the several members.

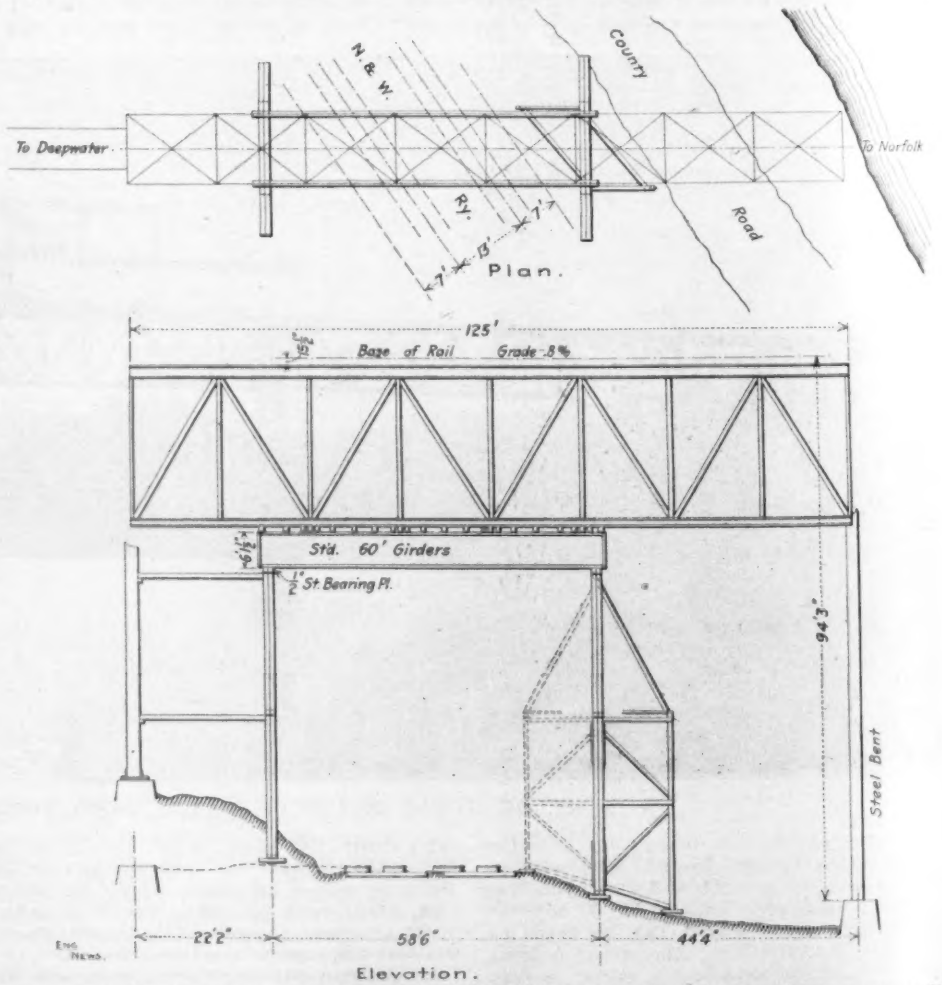


FIG. 2. COMBINATION FALSEWORK FOR ERECTION OF 125-FT. SHORE SPAN, GLEN LYN BRIDGE.

the top of the tower, giving the posts vertical support. Special extensions were bolted on to increase the length of the posts sufficiently to span the 30-ft. tower and reach a bearing against the bottom chords of the trusses. A pair of cast-steel bearing blocks, 2¾ ins. thick, was also provided for each chord, one block being made with a convex cylindrical face and the other made concave to fit it. These were to go between the abutting ends of the bottom chords, and were arranged to bolt to the ends of the chords. The cylindrical faces were set with their axes transverse to the trusses, thus allowing the necessary motion to prevent any excess bearing on the bottom of the castings due to the angular deflection of the chords as the erection proceeded and the deflection increased. The ends of all the bottom chords were milled to a true surface and these cast blocks were made with the connecting side slightly concave so that a bearing against the webs of the chords was assured.

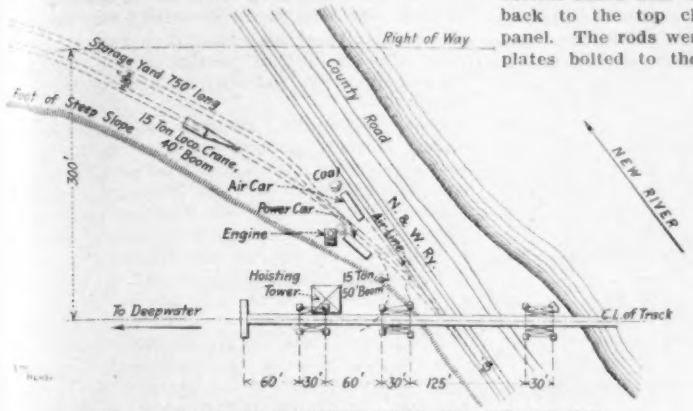


FIG. 3. PLAN OF STORAGE YARD ON WEST BANK.

TEMPORARY CONNECTIONS FOR OTHER SPANS.—The spans after the first required only a connection across the narrow space of 3 or 4 ins. which separates the adjoining ends of trusses over the piers. At the bottom chord the pair of bottom chords, but as all material had been ordered and the details practically made before the cantilever method of erection was finally accepted, it was decided to go ahead with the material as ordered. A pair of 2¼-in. round rods was therefore provided to engage the long bottom chord near the center and carry the load back to the top chord of the last completed panel. The rods were passed through skewback-plates bolted to the chords (Fig. 9) and were tightened up by double nuts at each end. As quickly as the web-members were placed beyond the center panel and the center top-chord section was connected up, the nuts were slacked off and the rods removed.

DISCONNECTING AND LOWERING UPON SHOES.—As each span was completed the outshore end was landed on a pair of 70-ton jacks set just inside the trusses and bearing against a special transverse girder bolted between the end posts (see Fig. 8). As soon as this jacking girder was bolted in place the pins connecting the shoes to the chords were driven. The end of the span was then jacked up to full reaction, by which operation the cast-steel blocks at the bottom and the ties at the top chord were relieved of stress. These erection connections being removed, the span was lowered to bearing on the masonry and the jacking girder was removed for use in the next span. Putting in place the lower panel of the permanent sway-bracing (in place of the jacking girder) completed the erection.

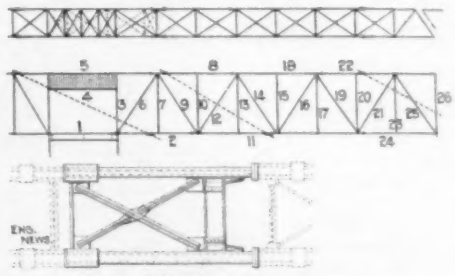


Fig. 4. Diagram of Erection of First River Span.
(Numbers indicate order of setting members.)

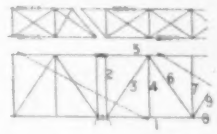


Fig. 5. Diagram of Erection of Second to Fifth River Spans.
(Numbers indicate order of setting members.)

ERECTION STRESSES AND CONNECTIONS.—In order to get the stresses with the greatest possible accuracy, each individual truss member was considered separately. A wind load of 30 lbs. per sq. ft. over the exposed area of one truss was assumed and all connection bolts were proportioned for the combined dead and wind loads. This assumption for the wind load was perhaps excessive, but no provision was made for shocks, and as a precautionary measure, to avoid the possible loss of camber from landing as a simple span, the remaining holes in the bottom-chord splices and main diagonal connections were filled with driftpins before jacking up the outshore end.

THE VERTICAL AND LONGITUDINAL DEFLECTIONS.—The deflection of the trusses as cantilevers had been figured originally to provide for the floor deck being about half in place. When it was decided to erect the trusses and land them without any superimposed load on the span no new calculations were made. The cast-steel bearing blocks at the bottom chords were made to an exact fit, while the ties for the top chords were shortened 1⅝ ins., enough to take care of 7 ins. of deflection at the end of the span, which of course had to include the deflection due to the stretch of the anchorage span as well as the cantilever. As a matter of fact the trusses all landed about 4½ ins. clear of the masonry, corresponding to a deflection of about 2½ ins. In jacking up the spans to remove the cantilever connections the actual height of lift necessary to reverse the stresses was not measured but was estimated by the foreman in charge as 3½ ins.

The most interesting feature of the erection from an engineering standpoint was the behavior of the bottom chords of the anchorage spans under compression. The maximum compressive stress in the bottom chords in the cantilever

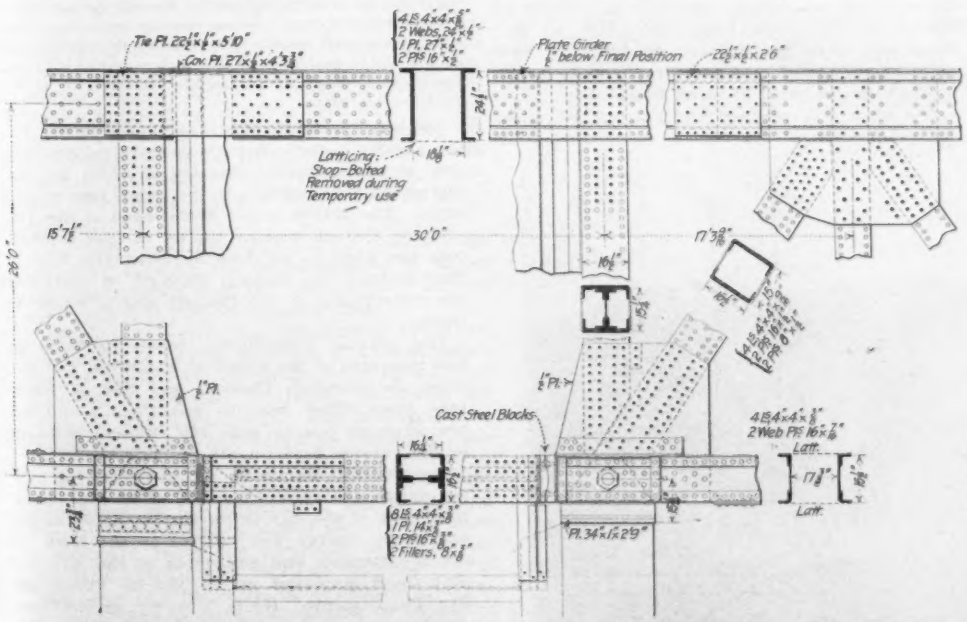


FIG. 6. DETAILS OF TEMPORARY TENSION AND COMPRESSION CHORDS ACROSS TOWER ON WEST BANK, FOR ERECTION OF FIRST RIVER SPAN.

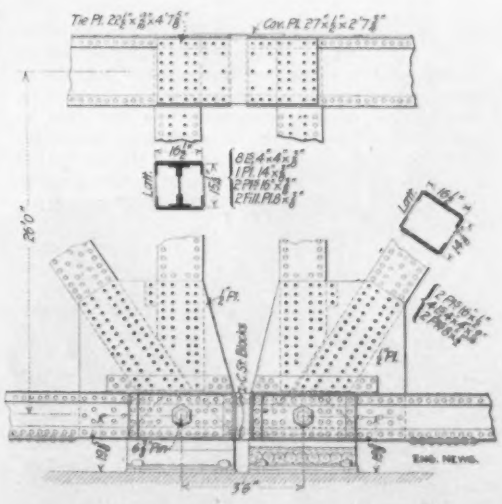
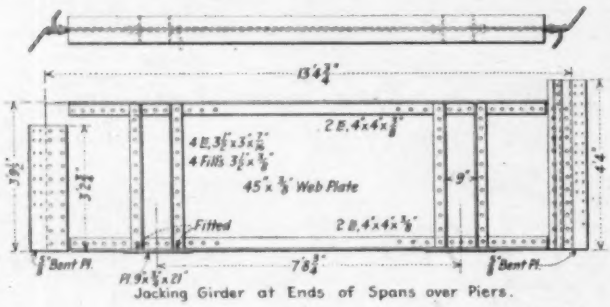


FIG. 7. DETAILS OF TEMPORARY SPLICES OVER PIERS FOR ERECTION OF SECOND TO FIFTH RIVER SPANS.

self-alining cast blocks was used, having been designed to fit both here and at the first span. At the top chord a set of splice-plates was bolted across between the adjoining chord ends (Fig. 7).

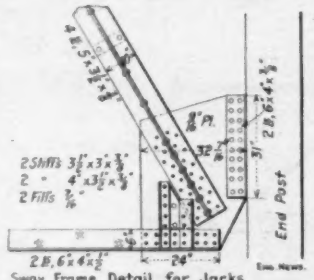
The diagonal rods shown in Figs. 4 and 5 were necessary to temporarily support the center section of bottom chord on account of the location of the splices. These temporary tie rods could have been avoided by changing the location of one splice and adding an additional one in the

condition for combined dead and wind loads figured about 12,000 lbs. per sq. in. Taking this stress as averaging 9,000 lbs. for the full length of the chord and adding the tension due to the dead load, there is an average change of stress of about 11,000 lbs. per sq. in., which with a coefficient of elasticity of 29,000,000 would produce a shortening of practically ⅝-in. in the bottom chord when the span, serving as anchorage span, changed gradually from the simple to



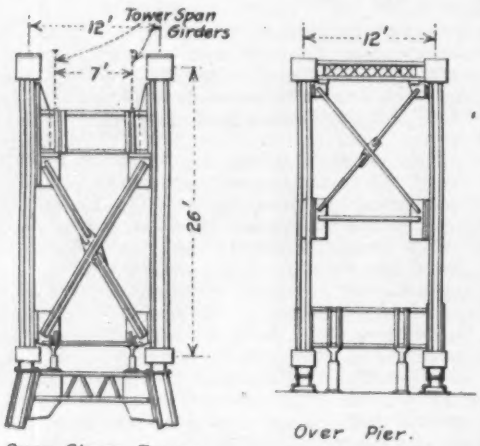
Jacking Girder at Ends of Spans over Piers.

A.



Sway Frame Detail for Jacks at End of Last Span.

C.



Arrangement of Jacks at Outshore Ends of Spans.

B.

A. Jacking Girder.
B. Arrangement of Jacks at Ends of First to Fourth River Spans.
C. Arrangement of Jacks at End of Last Span.

FIG. 8. TEMPORARY JACKING GIRDER USED AT OUTER END OF FIRST TO FOURTH RIVER SPANS, AND SKETCHES SHOWING PLACING OF JACKS.

the cantilever condition as the following span grew. This was a matter of no consequence in the last three spans, as the movable bearing of their respective anchorage spans was located at the inshore or west end and thus was free to move forward to suit the shortening of the chord. In the case of the 125-ft. span and the first river span, however, it was considered better designing to place the *fixed* shoe at the west or inshore end.

These spans being built that way, the precaution was taken to remove all bolts which connected the west shoes (the fixed shoes) to the columns, leaving the shoes free to slide forward while the spans were used for anchorages. It was considered that the reaction at the rear end would be relieved to such an extent that the shoes would move on top of the steel columns before the shoe of the span under construction would move on the masonry, being under a heavy load. As a matter of fact these rear shoes of the anchorage spans did move forward against the thrust in every case, an estimated distance of ⅜-in., indicating an average change of stress of about 6,700 lbs. per sq. in. in the bottom chords, a figure closely agreeing with the dead-load computations.

It is interesting to note that in both cases where the rear end of an anchorage span was a fixed end and rested on steel columns, the forward movement against the thrust of the cantilever span, or rather the shortening of the bottom chord due to the compressive stress, returned so evenly and accurately to proper condition that the bolts were returned to their places in the shoes without any drifting or jacking of the trusses whatever.

Moreover when the fifth and last span was completed the steel columns supporting it landed so nearly central on their respective pedestals that no adjustment whatever was made, the column anchor bolts being set just as the columns landed.

As already stated, the approach at the west end was erected during intervals between shipments of materials, months before the erection proper was started. On Oct. 3, 1908, the falsework for the 125-ft. span over the Norfolk & Western tracks was started; the first piece of steel was erected on Oct. 17. On Jan. 1, 1909, the tower on the east bank was completed and the last truss span landed on it—an elapsed time of 64 working days, of which 9 were lost on account of bad weather. The 1,140-ft. approach was started on Jan. 2 and the last span completed Jan. 23, 16 full days being worked. The time from starting the first truss span to the completion of the last approach was 82 working days, of which 11 were entirely lost on account of bad weather. The record is given here simply to make the description complete, as it is by no means unusual. The height to which all material had to be hoisted and the length of haul to reach the traveler—a quarter of a mile on the average—made rapid erection impossible.

The work was planned and carried out under the supervision of the regular engineering staff

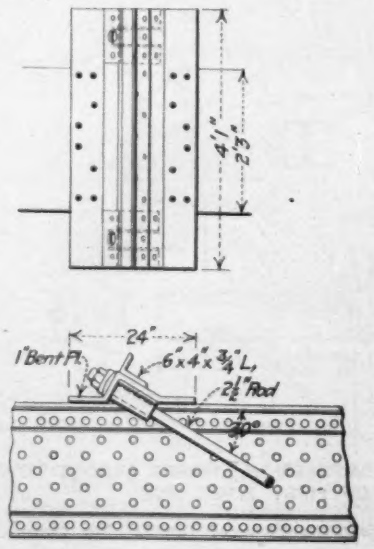


Fig. 9. Skewback Plate for Diagonal Rods Supporting Bottom-Chord Sections During Erection.

of the Virginian Railway, Mr. H. Fernstrom, M. Am. Soc. C. E., Chief Engineer, and Mr. F. F. Harrington, M. Am. Soc. C. E., Engineer of Bridges. The McClintic-Marshall Construction Co. was the contractor for the fabrication and erection of the superstructure.

A Municipal Manager at Staunton, Va.*

By R. D. HAISLIP.†

It must not be supposed that Staunton, Va., was not as well governed under the system in vogue before the appointment of a General Manager as any other town of its size in the country, as such was not a fact. Staunton's City Council was composed of high-minded, honest business men, who gave their service gratuitously for the common good, and often at very considerable sacrifice.

These councilmen, realizing that cities were merely business corporations in which the citizens received low taxes and good service in lieu of dividends, and therefore that cities should apply to their management the principles which purely commercial corporations have found to be effective, decided to try the experiment of concentrating the direction of the work in the various branches of the city government under one man, to be known as the General Manager, and to this end enacted the following ordinance:

Be it ordained * * * that there be appointed by the two branches of the Council in joint session as soon as possible after the adoption of this resolution, and thereafter annually at the regular election of city officers in July of each year, an officer to be known and designated as "General Manager."

The General Manager (except in the case of the first appointment under this resolution, which shall be until the next regular election of city officers in July, 1908) shall hold office for the term of one year and until his successor is duly appointed and qualified, unless sooner removed by the Council at its pleasure.

The General Manager shall be paid an annual salary of ..... dollars, and he shall have the right to employ one clerk at a salary of ..... dollars per annum, to be paid by the city, the amount to be hereafter fixed by the Council.

The General Manager shall devote his entire time to the duties of his office, and shall have entire charge and control of all of the executive work of the city in its various departments, and have entire charge and control of the heads of departments and employees of the city. He shall make all contracts for labor and supplies, and in general perform all of the administrative and executive work now performed by the several standing committees of the Council, except the Finance, Ordinance, School and Auditing Committees. The General Manager shall discharge such other duties as may from time to time be required of him by the Council.

The General Manager before entering upon the duties of his office shall execute a bond before the Clerk of the Council in the penalty of $5,000, with good and sufficient surety, conditioned for the faithful performance of the duties of his office.

Neither the Mayor nor the City Council, nor the various council committees have relinquished any of the authority invested in them by nor any of the responsibility to the people who elected them. The Mayor is the official head of the city, and the Council assigns to the various committees the work to be done by them and they in turn instruct the General Manager to carry out the instructions of the Council and to report to them.

The regular committees, consisting each of two members of the Board of Aldermen, and two from the Common Council, any two of whom (one from either branch) constitute a quorum are advisory boards with the General Manager in the various departments.

The General Manager's duties consist of having entire charge and control of all the executive work of the city in its various departments, and entire charge and control of the heads of the departments and employees of the city, except those employed in the School, Police and Fire Departments. He is, however, general purchasing agent for all the departments. He makes and signs all contracts for the city, subject to the approval of the City Council. He is City Engineer and issues building permits and licenses to milk dealers. In other words, he occupies the same position as that of the general manager of any business corporation, reporting quarterly in detail to the Mayor and Council just as the general manager of a business corporation would report to the president and board of directors.

A modern system of voucher-bookkeeping has

*From the "Municipal Economist," January, 1909.
†Editor "Dispatch," Staunton, Va.

been inaugurated, by which all accounts can be kept up to within 24 hours of any transaction, the information being procured from daily work reports furnished by the heads of the various departments, showing labor and material used on each piece of work.

The purchasing is done on requisition from the department heads, which, if approved, are filed as records, and an office requisition issued to the merchant showing to which department the goods must be charged, a duplicate of which is made and kept in the office as a record of purchases, thereby enabling the exact financial standing of any department to be ascertained in a few minutes. All bills against the city are made on forms furnished by it and must be filed with the General Manager before the 25th of each month and are dated as received. These bills, after being checked and entered upon the department books to which they belong, are audited by the Auditing Committee and payment authorized by the City Council.

All accounts are handled in this manner except freight bills and those upon which a cash discount is allowed, which are paid by the Treasurer, after being approved by the General Manager.

Not only does this office keep a complete set of books showing the disbursements in all of the departments and the collections from the same, but it is also an auditing office.

Before this system was inaugurated the council committee having direction of a department, usually through the chairman, after consultation with the committee, undertook to direct the work of the department. Not receiving any compensation for his work, the chairman naturally could devote only a small part of his time to this work. He was supposed to direct the purchasing of his department, but in practice the paid superintendent under him or some other subordinate, made the purchases and consulted the department pretty much as he pleased. The bills came in irregularly, often months after the purchase had been made, making it impossible to keep the accounts of each committee in a way that a correct balance could be obtained, there being no system as to recording the purchases, nor the bills that had been paid, which were merely filed, the only check on them being the memory of the purchaser.

There was no cooperation among the departments, no correlation of forces, no general supervision, and drawing together except at the regular monthly meetings of the Council, when the consideration of a multitude of matters, mostly of a legislative character, prevented any real unifying of the business.

The keen business men on the Council were always aware that something was needed; the large revenues of the city were being spent, but small results were being obtained.

They studied the subject carefully and reached the conclusion that too many people were doing the buying and spending, and the directing of work, and that the remedy was to place the entire business of the city under one head and look to him for results.

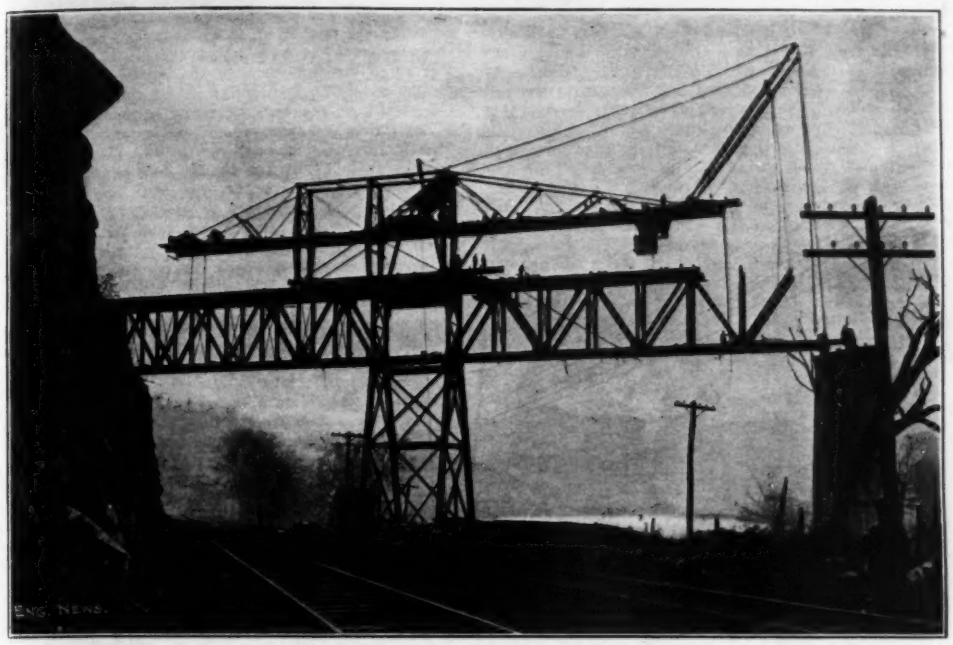


FIG. 10. ERECTING LAST THREE PANELS OF FIRST RIVER SPAN, GLEN LYN BRIDGE.

Several measures were drawn and offered from time to time, the Galveston-Houston idea forming the basis; they were discussed at repeated meetings and through the press, and finally at the end of several months of agitation and study the ordinance establishing the office of General Manager was enacted, signed by the Mayor and became a law.*

*We learn from another source that the Commission plan could not be adopted without amendment to the State Constitution, but that there was legal authority for creating new city offices and prescribing the duties of the incumbents.—Ed.

The incumbent of this office, Mr. Charles E. Ashburner, a civil engineer with a varied experience, has coordinated all the working departments of the city, unifying and systematizing the purchasing and establishing thorough cooperation among all the departments. By the system introduced he has been enabled to transform many parts of the city that had a rundown appearance. The system has been in every direction promotive of economic results. The city is coming nearer getting a dollar's worth for every dollar expended than ever before in its history. The business men of the Council are pleased with the results.

Considering the change from the popular standpoint it may be said that the people generally are very much pleased.

There is one feature about the system that seems to have great merit, even if it costs something extra, which it does not: every citizen who has a complaint or suggestion to make, no matter what about, whether rubbish in a street or alley, or lot, a nuisance of any kind, a bad place in the pavement or street, some need in water or light, knows where to take his complaint and receive a respectful hearing, and obtain a remedy if practicable. He does not need to go to one man and be referred to some one else, and then somebody else, till he becomes discouraged and finally reaches the conclusion that it is hopeless to complain or to suggest. The General Manager notes the complaint, investigates usually personally, and if there is a remedy he places the duty of supplying it on the proper person.

The advantages of the system, from money standpoints, are so obvious that it is not worth while to go much into details, but there is one other to which I would like to refer: the nature of the duties of the General Manager takes him every day or so over the whole city. He becomes familiar, as no other person does, with every nook and corner, and if he is observant and foreseeing and farseeing, as a man in this position should be, he will naturally conceive of many changes and improvements that if made will tend to harmonize and develop the city along aesthetic as well as practical lines, and he will bring forward for discussion and study on the part of the city's legislative body suggestions of the highest value.

As a matter of fact, Staunton's General Manager has already performed invaluable service to the city along these very lines.

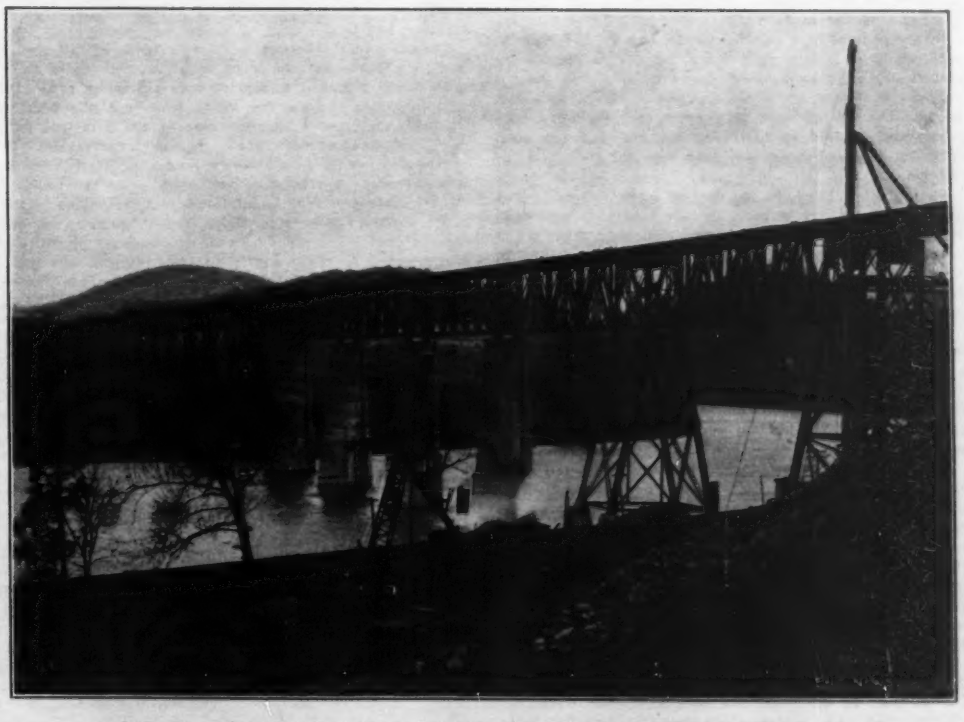


FIG. 11. NEW RIVER BRIDGE AT GLEN LYN, VA., AFTER COMPLETION.

Reinforced-Concrete Caissons: Their Development and Use for Breakwaters, Piers and Revetments.*

By W. V. JUDSON,† M. Am. Soc. C. E.

For some time work has been progressing in the Milwaukee district which has involved the study, design and construction of reinforced-concrete caissons for various structures. This system of construction has proved so efficient and economical, and the studies have been so promising that a full account of the work seems desirable.

No dry dock is available in which to construct the caissons, and the development of the latter has been influenced by the fact that the caissons must be built on ways and launched therefrom like any vessel. The heaviest caissons launched have weighed about 119 tons each, but caissons weighing 352 tons each will be built and launched at Milwaukee next summer, and there is no reason to doubt that much heavier caissons can be launched with perfect facility.

The ways that have been employed consist each of three 12 × 12-in. stringers, descending into the water with a slope of one on ten. With this slope the caissons start of themselves and do not bind. The stringers are supported on capped pile bents, extend far enough back to give building space, and terminate about 6 ft. under water.

The caisson is begun upon a plank floor, 3 ins. or 4 ins. thick, which is launched with and forms a part of the caisson. Spikes are driven through the plank so as to project upward into the concrete, and building paper is pressed down upon the plank to reduce the leakage. A temporary wooden deck is used while launching and while towing any considerable distance in the open lake. A caisson is designed so as to have its width and depth proportionate to the work it is to do in a breakwater or pier. Its length is limited partly by the undesirability of too great aggregate weight, but more by the consideration of longitudinal strength, considering the whole caisson as a beam upon which various stresses may be brought to bear under different assumptions as to character of foundations. The reinforced-concrete walls, bottom and cross-walls are limited in thickness by considerations of buoyancy, but within the limits of buoyancy it is practicable to design with reasonable economy.

Whether in a given caisson to employ thick walls and few chambers or thin walls and many chambers is a question, in solving which careful judgment should be employed. With thick walls and few of them the cost of putting the reinforced concrete in place is reduced. On the other hand with more numerous walls the spans are reduced and the caisson even with thinner walls sometimes is a safer structure. In some cases it would probably be advantageous to substitute interior bracing for some of the cross walls.

In the general case small caissons, say 14 ft. square in plan, such as I have used for a number of light-house structures, must be monocellular, for the reason that, if they were divided up, considerations of buoyancy would require the use of such thin walls that excessive care would be required in their construction, and the steel if sufficiently embedded would be too near the center of the walls.

On the other hand, in the case of large caissons, such as one 60 ft. square in plan, it is best to have numerous chambers, inasmuch as the cross walls, even if there are many of them, may still be of reasonable thickness. It is not believed that in many cases it would be true economy to use caissons which would be lighter in weight than conditions of buoyancy require.

STRESSES.—The stresses brought upon a caisson are ordinarily as follows: Those due to water pressure upon the outside; those arising from excess of pressure, due to the filling, outward; those due to bending, if the foundation be imperfect and the caisson be supported, for example, under its two ends, or under its middle; and those due to wave action.

When the caisson is launched it may be but ten days old, and the concrete relatively green. When launched, unless long and expensive ways are employed, the caisson plunges low in the water, and the pressure due to the latter is at a maximum.

It is generally found that if the walls and the steel therein be proportioned to care for this initial pressure safely, then the other stresses are negligible, although investigation should always be made as to the sufficiency of the longitudinal strength.

In designing caisson walls and bottoms, several methods have been employed. For a time the steel near the surfaces of the walls has been assumed to take up all the tension and compression, the neutral axis to be in the center of the wall, and the concrete to serve only as the web of the beam. Later the more correct and economical theory has been applied, that the steel on the tension side takes up all the tension, while the steel on the other side reinforces the concrete in compression from the neutral axis up. The horizontal steel reinforcement employed in the walls has been distributed in equal amounts on each side of the neutral axis.

Deformed bars are preferred, as plain bars might not form a sufficient bond with the green concrete.

From study and experiment it is believed that the horizontal steel rods in a wall may be proportioned as follows: Assume a reasonable thickness for the wall dependent upon flotation desired, etc., say from 12 to 16 ins. Consider the wall as a series of discontinuous beams, one above the other, each with a distributed load corresponding to an assumed maximum water pressure. Take the span in the clear without allowance for the filling in of the corners where walls meet (this gives a factor of safety). Neglect the facts that the wall is a slab, due to the presence of vertical rods, and that it is a continuous beam (these give additional factors of safety). Then proportion the steel in accordance with the formula* stated on p. 117, Proceedings of the Am. Soc. C. E. for February, 1909, allowing the steel in tension to be strained to 18,000 lbs. per sq. in. Mr. Howard A. Hoeing of this office has prepared from these formulas diagrams by means of which a graphical solution of any ordinary problem of this character can be quickly made.

In the walls a certain number of rods are introduced vertically. They have generally been placed in pairs, spaced 2 ft. apart. The vertical rods add somewhat to the strength of the walls, and reinforce against diagonal tension, considering the caisson as a single beam. They also serve as supports for the horizontal rods in forming.

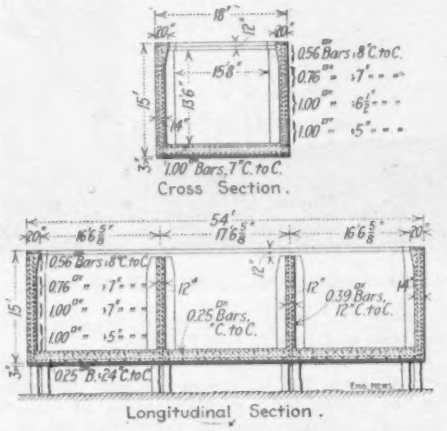


Fig. 1. Reinforced-Concrete Caisson for South Pier, Milwaukee, Wis.

The steel in the bottom is determined as in the case of a wall. If the caisson is to be placed on a pile foundation, the principal system of rods ordinarily runs longitudinally to support the filling from one pile bent to another. For use on an enrockment the principal rods in the bottom are ordinarily placed transversely. In the bottom, as in the walls, the rods are placed in pairs, one rod near each surface, and again as in the walls, a lighter system of rods is employed at right angles to the main system. Wherever walls come together, the reentrant angles are filled in with concrete and short rods are inserted in these angles to stiffen them.

Inasmuch as no records were available where beams with equal reinforcement top and bottom, and of concrete but ten days old, have been tested to destruction a few tests of this character have been made. The results of these tests confirm the belief that the method of designing above described will give reasonable factors of safety, which will be increased somewhat by other considerations that have already been mentioned. The experimental tests indicate that failures due to compression in the concrete need not be apprehended except where the percentage of reinforcement is very large, larger in fact than would ordinarily be employed in caissons following the method of designing proposed. Where the shear is great, however, there should be an investigation as to the sufficiency of the resistance to diagonal tension stresses. Considering all of the conditions and the results of the tests, it would appear that at time of launching compressive stresses on extreme concrete fiber up to 1,000 lbs. per sq. in. and diagonal tension stresses up to 50 lbs. per sq. in., *both as determined by the formula where the caisson walls are designed as herein proposed* would not be excessive.

At the time the caissons are launched, any failure would probably result from failure in the green concrete in portions of a wall where the percentage of steel is greatest, or from tension failures in the steel where the percentage of reinforcement is small. But as the concrete rapidly gains in strength, such designing as I have described would make the walls, after the lapse of time, much stronger, and where the percentage of steel is greatest, approximately equally liable to yield through tension or compression failures. It appears therefore advisable to use steel with a high elastic limit, the gain being most noticeable on portions of a wall where the percentages of steel are relatively small.

A method employed for investigating the longitudinal stiffness of a caisson is shown in Fig. 2. The 54-ft. Milwaukee caisson shown in Fig. 1 has been chosen for purpose of illustration. As this caisson is designed to be supported on four bents of piling, the assumption is made that one of the end bents is cut off too low to perform its function. One-third of the caisson would then be unsupported and would act as a cantilever, bringing the steel near the top of the caisson into tension. Assuming that the concrete serves no function except to unite the steel rods and make them act together, the steel is projected into a beam, under the assumption that all the steel in a given horizontal plane is united to form part of a flange or web. The maximum bending moment is applied to this hypothetical beam and the tension in the extreme fiber thus determined is assumed to be the tension in the steel rods near the top of caisson.

No difficulty has been experienced in securing watertight walls. The caissons are constructed of 1:2:4 concrete and the latter is ordinarily dense enough without treatment. However, the caissons, on the ways, have as a rule been partially filled with water, to test watertightness, and in some cases a grout wash, containing lye and alum, has been applied to the exterior.

It is believed that the harmful effects of salt water would be reduced to a minimum in the case of the dense concrete of the caissons.

One of the first thoughts that arose, when the permanent filling of the caissons was under consideration, was as to the effect of interior freezing. In the autumn of 1907 a small caisson 7 ft. long, 5.81 ft. wide, and 5.67 ft. high, with bottom and walls varying in thickness from 4 to 5 ins., was built on the Government dock at Milwaukee, Wis., tumbled off the dock into the water, thus falling 6 ft., towed to the mouth of the harbor, beached in 3.8 ft. of water in an exposed location on North Point, filled with sand, and paved over with 6 ins. of concrete. This caisson, which is still in place, has not moved perceptibly nor suffered damage from any cause whatever.

THE ALGOMA BREAKWATER.—The first caissons for actual service were built by day labor at Kewaunee Harbor, Wis., during the summer of 1908. They were used in constructing a breakwater at Algoma Harbor, Wis.*

The caissons, 24 ft. long (or 25 ft. long with the guide timbers), 15 ft. wide and 12 ft. 4 ins. high, with walls 12 ins. thick, one cross-wall 10 ins. thick and a bottom 16 ins. thick (not including the 4-in. thickness of plank bottom), weighed 119 tons each in air. They were constructed on ways, of which three sets were provided, at the rate of one every third day after organization was completed. There were no difficulties encountered in any part of the work. The caissons, temporarily decked, were launched and towed 12 miles in the open lake to Algoma Harbor, Wis., where the decks were removed and the caissons parked until they could conveniently be placed in the work. When launched, as the ways terminated but 6 ft. below datum, the caissons tilted forward until the decks made an angle of about 30° with the water surface, and sunk until they were just submerged. They immediately righted themselves and floated with a draft of 9.7 ft., corresponding to a freeboard of 2.63 ft.

At Algoma the arm of the breakwater constructed of

*These caissons were described and illustrated in Engineering News, Oct. 15, 1908, p. 421.—Ed. Eng. News.

*These formulas are a part of the Progress Report of the Joint Committee on Concrete and Reinforced Concrete, and are as follows:

$$k = \sqrt{2n\left(p + p'\frac{d'}{d}\right) + n^2(p + p')^2} - n(p + p').$$

$$z = \frac{\frac{1}{2}k^2 d + 2p'n d'\left(k - \frac{d'}{d}\right)}{k^2 + 2p'n\left(k - \frac{d'}{d}\right)}$$

$$j d = d - z.$$

$$fs = \frac{M}{pjbd^2}$$

in which k = ratio of depth of neutral axis to effective depth, d.

$n = \frac{Es}{Ec} = \frac{\text{Modulus of elasticity of steel}}{\text{Modulus of elasticity of concrete}}$

p = steel ratio for tension
p' = steel ratio for compression steel
d = depth to center of tension steel
d' = depth to center of compression steel
b = breadth of beam
z = depth of resultant compression below top
j = ratio of lever arm of resisting couple to depth d
fs = unit tensile stress in steel
M = moment of resistance.

In the above the values of n, fs and b are constant, $p = p'$. —Ed. Eng. News.

*Paper presented to the Western Society of Engineers, May 19, 1909.

†Major, Corps of Engineers, U. S. Army; Engineer Commissioner, District of Columbia, Washington, D. C.

caissons was 511.7 ft. long. The natural depth varied from 15.3 to 13.5 ft. The breakwater faces approximately southeast by south and is exposed to a fetch of about 140 miles measured normally to the breakwater.

At Algoma the caissons were sunk on a pile foundation so as to project one foot above datum when placed. Intermediate between the pile tops and the plank bottoms of the caissons were cast iron disks, covering the pile tops and placed upon them, of course, before the caissons were sunk. These disks were cast with waffle patterns on each side, so that the corrugations, being pressed down into the piles and up into the plank bottom, would introduce a desirable amount of friction and at the same time somewhat equalize the loads on the piles. The initial sinking was by means of water, introduced through siphons. The caissons, except several that were filled otherwise for experimental purposes and which will be hereinafter described, were then filled with broken stone to within 4 ft. of the tops. The rest of the filling was of lean concrete, and the breakwater was completed by riprapping its sides and adding a concrete superstructure.

I cannot speak too highly of the services of Mr. L. E. Lion, Assistant Engineer, who was in local charge of the work. That this undertaking, when so much was novel, progressed so smoothly was largely due to his skill and careful foresight.

COSTS.—Before the Algoma breakwater was commenced it was estimated that its cost, if built of stone-filled wooden cribs of the type usually employed on this lake, the same being placed on pile foundation and capped with a standard concrete superstructure, would be $105.18 per lin. ft., and this was the cheapest way, following former practice, in which the breakwater could have been built in permanent form. The estimate for the caisson breakwater was $103.74 per lin. ft., large unit prices having been purposely assumed because of the novelty of the work. The actual cost of the work proved to be but $75.67 per lin. ft., although there should be added to this, to make the estimates fairly comparable, $2.62 per lin. ft. to allow for the saving on riprap stone that was obtained from the demolition of the old south pier. It therefore appears that the saving on the Algoma breakwater, due to the use of the caissons, was $26.89 per lin. ft., or 25.57%.

Incidentally it may be said that for the construction of the harbor at Algoma there was available on Jan. 1, 1908, $44,822 cash, with $100,000 additional authorized and subsequently appropriated. This was considered but a fair allowance for the work projected, if done by contract, and in fact the prices bid by contractors, after formal advertisement, were so large that the cost would have exceeded the amount available. Using Government plant and days labor, the harbor is now completed, except for dredging 20,000 cu. yds., more or less, and depositing about the pile pier 700 tons, more or less, of riprap, and after this small amount of work is completed in the spring there will remain of the amount originally available at least $45,000.

The following is a summary of the cost of the caisson breakwater, on a unit price basis:

Cost of the 20 caissons up to delivery at Algoma, Wis.

Item 1.—Cost per cu. yd. of reinforced concrete—		
Plant charge, including cement warehouse		$0.714
Charge for launching ways		0.750
Charge for forms		0.908
Materials:		
Cement	$1.549 per cu. yd.	
Sand and stone	1.298	
Steel reinforcing bars (including wire and spacers)	4.914	
Timber in bottoms, decks and guide timbers	1.275	
Nails, spikes and bolts	0.522	
Fuel & miscellaneous supplies	0.225	$9.783
Labor, including superintendence		5.083
Towing 12 miles		0.423
Cost per cu. yd		$17.661
Total cost of the 20 caissons, containing 1,045 cu. yds. of reinforced concrete		$18,456.75

Work at Algoma after caissons were delivered.

Item 2.—Cost of 820 cu. yds. 1:5:10 concrete filling—		
Materials and labor:		
Cement, 0.735 bbl. per cu. yd., at $1.15		$0.845
Aggregate, 1.001 cu. yds., at $1.25		1.251
Handling cement, per cu. yd. of concrete		0.033
Handling aggregate, per cu. yd. of concrete		0.314
Labor and superintendence, per cu. yd. of concrete		1.295
Fuel, per cu. yd. of concrete		0.189
		$3.927
Plant charge, including warehouse		0.890
Forms		0.030
Total per cu. yd		$4.847
Total for 820 cu. yds.		$3,974.54
Item 3.—Cost of 421 cu. yds. 1:3:5: concrete superstructure—		
Materials and labor:		
Cement, 1,264 bbls. per cu. yd. of concrete, at $1.15		$1.454
Aggregate, 0.908 cu. yd. per cu. yd. of concrete, at $1.25		1.248
Handling cement, per cu. yd. of concrete		0.059
Handling aggregate, per cu. yd. of concrete		0.350
Labor and superintendence, per cu. yd. of concrete		2.660
Fuel, per cu. yd. of concrete		0.189
		5.960
Plant charge, including warehouse		0.890
Forms		0.525
Total per cu. yd		$7.384
Total for 421 cu. yds.		$3,108.66

Item 4.—Pile foundation, 376 piles, aggregating 15,040 lin. ft.—

Cost of piles	$2,481.60
Cast-iron pile disks	176.40
Labor and superintendence; sharpening, driving, cutting off and leveling pile foundation.	1,261.34
	$3,919.34
Plant charge	320.00
Total	$4,239.34
Cost per pile, 4,239.34 ÷ 376 = $11.275.	
Item 5.—Riprap stone—	
Large stone purchased, 1,572.45 tons, at $1.19	$1,871.22
Small stone purchased, 3,825.52 tons, at $0.98	3,749.01
Riprap from demolition, 1,231 tons, at $0.25	307.75
Labor and superintendence placing purchased riprap	1,718.24
Time of dredge casting riprap	124.41
Plant charge, scows and derrick	264.00
Total cost 6,628.97 tons of riprap in place	$8,034.63
Cost per ton, 8,034.63 ÷ 6,628.97 = $1.212.	
Item 6.—Miscellaneous—	
Setting ranges and preliminary work	$71.81
Removing decks and preparing caissons for sinking	329.07
Towing with gasoline launch	323.25
Miscellaneous supplies	181.65
	$905.78
Total cost of 511.7 lineal feet of breakwater; aggregate of Items 1 to 6 inc.	$37,718.58
Cost per lin. ft. of completed breakwater.	$75.67

Having thus stated in detail the cost of the Algoma work it is proper to study the figures with a view to ascertaining how estimates for future work of a similar character may be based upon them.

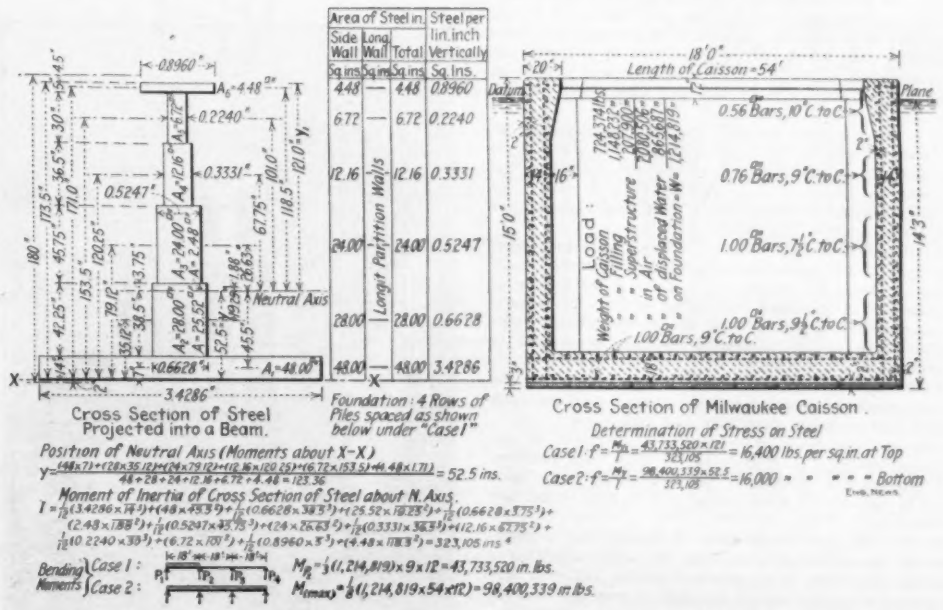

Area of Steel in
Side Wall
Long. Wall
Total
Steel per lin. inch Vertically
Sq. ins.
4.48 — 4.48 0.8960
6.72 — 6.72 0.2240
12.16 — 12.16 0.3331
24.00 — 24.00 0.5247
28.00 — 28.00 0.6628
48.00 — 48.00 3.4286
Longit. Partition Walls
Neutral Axis
Cross Section of Steel Projected into a Beam.
Foundation: 4 Rows of Piles spaced as shown below under "Case I"
Position of Neutral Axis (Moments about X-X)
y = 52.5 ins.
Moment of Inertia of Cross Section of Steel about N. Axis.
I = 323,108 ins⁴
Bending Moments Case 1:
Case 2:
M = 43,733,520 in. lbs.
M = 98,400,339 in. lbs.
Length of Caisson = 54'
Datum
Plane
0.56 Bars, 10" C. to C.
0.76 Bars, 9" C. to C.
1.00 Bars, 7½" C. to C.
1.00 Bars, 9½" C. to C.
1.00 Bars, 9" C. to C.
Load:
Weight of Caisson
Filling
Superstructure
in Air
of displaced Water
on Foundation = W =
Cross Section of Milwaukee Caisson.
Determination of Stress on Steel
Case I: f = 16,400 lbs. per sq. in. at Top
Case 2: f = 16,000 " " " " Bottom

FIG. 2. METHOD OF COMPUTING STRESSES IN STEEL IN CAISSON SHOWN IN FIG. 1.

First, it is to be remembered that the working hours were eight per day, except that during the favorable working months of July, August and September, by Executive Order, the hours on Saturdays were reduced to four, although payments were made for full time.

Second, the harbor at which the caissons were built was 12 miles distant from the harbor where they were used. Under more favorable conditions in this respect the towing charge would be much reduced. Moreover, as the caissons were to be towed from one harbor to another in the open lake the temporary decks employed were more numerous, and the expense of placing the decks (calking, etc.) was greater than would have been the case if a deck had been needed when launching only. In the case of larger caissons the timber bottoms, per cubic yard of concrete, would also be much reduced, and it is proposed hereafter to omit the guide timbers at the ends of the caissons.

Third, the total volume of the Algoma work was small, but 511.7 lin. ft. of breakwater, containing 1,045 cu. yds. of reinforced concrete, having been constructed, although 22 caissons, containing 900.62 cu. yds., were built for Manitowoc Harbor during the same season and with the same ways and plant, thus reducing the charges for these items. The Algoma caissons, and the Manitowoc caissons as well, were small compared with others projected. The walls and bottoms were relatively thin, and for a given expenditure they did not run rapidly into "yardage." It is probable that larger caissons, employed to build more considerable lengths of breakwater, would cost considerably less per cubic yard for plant, forms, ways and labor.

Fourth, in the case of the Algoma caissons 248 lbs. of steel per cu. yd. of concrete was employed. Careful study has shown that in the case of the larger caissons with cross-walls suitably spaced the steel can be safely reduced by a considerable amount.

All of these considerations lead one to believe that, where the amount of work is sufficiently large, the cost of the reinforced concrete of the caissons in place in the finished work may be estimated under conditions now obtaining in this vicinity at about $16 per cu. yd., inclusive of all materials, labor, plant and contractor's profit. But it is doubtful whether contractors will take their first work of this character, on account of its novelty, at figures quite so low.

Five of the Algoma caissons were filled in experimental ways for the sake of securing information. The 16th and 17th caissons from the northerly end were filled with sand to within 4 ft. of the top, and the upper 4 ft. with 1:5:10 concrete which was supported by posts extending through the sand to the bottoms of the caissons. The 18th caisson was filled with sand to within 2 ft. of the top, and the upper 2 ft. with 1:3:5 concrete. No supporting posts were used in this caisson. In the 19th caisson sand was used for filling to within 2 ft. of the top, the upper 2 ft. being filled with 1:3:5 concrete, which was supported by posts. The 20th caisson was filled with riprap stone to within 1 ft. of the top, and the upper foot filled with 1:3:5 concrete supported by posts.

The first caisson for Algoma was molded on June 1, 1908, and the last on Aug. 24, 1908.

OTHER CONSTRUCTION.—With the same plant, at Kewaunee Harbor, the construction was immediately begun of a number of caissons required for use at Manitowoc Harbor, Wis., 29 miles distant. By this time the force was so well organized that it was found desirable to build one caisson every other day. To secure working space, caissons four days old were moved part way down the ways without difficulty. The Manitowoc caissons are 24 ft. long, 14 ft. wide and 11 ft. 4 ins. high. The bottoms are 18 ins. thick (including 4 ins. of timber). The walls are 10 ins. thick and the cross-wall 8 ins. thick. Twenty-two caissons were built for Manitowoc, of which one was of "special" shape. Eleven of these caissons were towed to Manitowoc and parked for the winter in the river; the rest were stored at Kewaunee.

Two special caissons still remain to be built for the Manitowoc work, which will include about 580 lin. ft. of concrete caisson breakwater, completing the projected harbor there. Next spring the Manitowoc caissons will be sunk on a riprap foundation and filled with lean concrete. Eventually they will receive a concrete superstructure, but not before they have stopped settling.

A contract has recently been let for the reconstruction of the south pier at Milwaukee Harbor (Fig. 1). The outer 216 lin. ft. of this work will consist of four caissons, each 54 ft. long and 18 ft. wide. The bottoms will be 21 ins. thick, including 3 ins. of planking. The side and end walls will be 14 ins. and the two cross-walls each 12 ins. thick. These caissons will weigh 352 tons each. The next 360 ft. of pier will consist of ten caissons, each 36 ft. long and 15 ft. wide. The bottoms will be 19 ins. thick, including 3 ins. of planking. The side and end walls will be 12 ins. and the two cross-walls each 10 ins. thick. These caissons will weigh 175 tons each.

The Milwaukee caissons will be built on and launched from ways located at Milwaukee belonging to the United States. They will be sunk on pile foundation, as dredging must be done close up to them. The United States will furnish the piles and the reinforcing rods. The

contract prices are $7.50 each for driving and cutting off the piles; $15.50 per cu. yd. for reinforced concrete in place; $1.45 per ton for filling stone and riprap; and $6 per cu. yd. for lean concrete filling.

The caissons thus far described have had vertical walls of uniform thickness from top to bottom, except that the walls of the Milwaukee caissons are to be thickened somewhat at the top, the better to hold the steel needed for longitudinal stiffness, and to avoid weakening the caissons at the top where timber is ordinarily let into the concrete for the purpose of holding the decks.

It is obvious that caissons with walls inclined inward from bottom to top would better resist wave action. Per cubic yard of contents they would possess greater moments of stability against overturning and at the same time the overturning moment due to wave action would be lessened, inasmuch as the wave pressure would act normally to the inclined front surface, and thus have a reduced lever arm.

It is also obvious that, as the walls are ordinarily subjected to the greatest pressure at points near the bottom, they may to advantage be made thicker near their bases.

To meet typical conditions in breakwater construction and the like a number of caissons have been designed.*

The use of reinforced-concrete caissons for breakwaters, bulkheads and the like appears to be undergoing development simultaneously in Europe and in America. Press reports state that at Kobe, in Japan, caissons are to be built 120 ft. long, 26 ft. wide and varying from 32 to 41 ft. high, to form the quay walls of a new harbor. It is proposed to build these caissons on timber stagings, from which they will be lifted by a special depositing dock, taken into deep water, allowed to float off the dock by sinking the latter, and then towed to their destination.

Similar but smaller caissons are being employed at the port of Barcelona. These caissons are put afloat, it is understood, by means of a depositing dock similar to that mentioned above. In some cases, probably in most cases in which caissons have recently been used abroad, they have been built in dry docks.

In recent technical publications there have been brief accounts of the employment of reinforced-concrete caissons for docks at Amsterdam, and for a breakwater at Touapsé. The published Proceedings of the International Congress of Navigation, held at St. Petersburg, 1908, contain most of the information that is now available in America on this subject.

Briefly speaking, the many advantages afforded by the use of reinforced-concrete caissons include economy, both initial and ultimate, durability and sightliness.

The great stiffness and substantial character of the caissons render it practicable to tow them long distances indepeudent of conditions of the sea. Many risks, incident to the use of other caissons or wooden cribs, are eliminated.

When over their foundation, if desired, the caissons may be quickly filled with water by using a sufficient number of siphons, or by means of valves, and with this contained water the caisson becomes at once safe against moderate seas. When sunk the tops of the caissons project above the water surface and the walls form cofferdams so that the chambers may be successively unwatered and the filling put in dry if desired.

When a timber crib is capped with a concrete superstructure it is necessary to employ blocks to build up from a plane say 2 ft. below the water surface to a plane say 2 ft. above the latter. It is difficult to provide a bond between the blocks and the timber, so that the monolithic concrete placed on the blocks must be massive with a view to holding down the blocks by weight alone. In the caissons, the upper parts thereof correspond to the blocks, and are of course solidly bound to the structure beneath. Hence a much less massive superstructure is required.

During construction the plant may be centralized. After a caisson is built gravity carries it down the ways and makes clear the working place. The construction of a piece of work employing a considerable number of similar caissons lends itself to organization and consequent economies that ordinarily result only from the favorable conditions under which factory work is prosecuted.

The cost of timber is rapidly appreciating, and the time when temporary structures were good enough is passing. There is probably much to be learned about reinforced-concrete caissons, and much educational work to be done among contractors, but apparently in no field will reinforced concrete make greater strides in the immediate future than in such works as breakwaters, piers, dock-fronts, revetments, bridge piers and similar structures where reinforced-concrete caissons may be employed.†

*In the original paper Maj. Judson describes, with drawings, a number of designs for various kinds of marine work.

†On account of lack of space we are unable to print the following appendices to Maj. Judson's paper:

Appendix I.: Diagrams for Reinforced-Concrete Beams Having Equal Reinforcement Top and Bottom; by Howard A. Hoeing.

Appendix II.: Description of Tests of Reinforced-Concrete Beams; by J. A. B. Tompkins, M. Am. Soc. C. E.

Appendix III.: Description of Algoma Caisson Work; by L. E. Lion, M. Am. Soc. C. E.

Reinforced-Concrete Torpedo-Testing Station in the Bay D'Hyeres, Near Toulon, France.*

[Contributed.]

The structure described in this article is a large, cellular reinforced-concrete pier, built in one of the bays of the Mediterranean as a testing station for torpedoes. Aside from the function of the station, which is somewhat out of the ordinary, the main novelty is in the construction of the superstructure, which was built as a reinforced-concrete caisson on shore and towed, under its own buoyancy, some 20 miles to its permanent foundation. In this respect this "Batterie des Maures," as it is called, is similar to the reinforced-concrete caissons built for the breakwater at Algoma, Wis., and described by Maj. W. V. Judson, U. S. A., in Engineering News, Oct. 15, 1908, pp. 419 and 421, and on p. 34 of this issue.

At the Toulon Arsenal the French Navy Department has been building torpedoes of the Whitehead type for sometime past. Recently, however, the firm of Schneider & Co., armament manufacturers, proposed to furnish the navy with

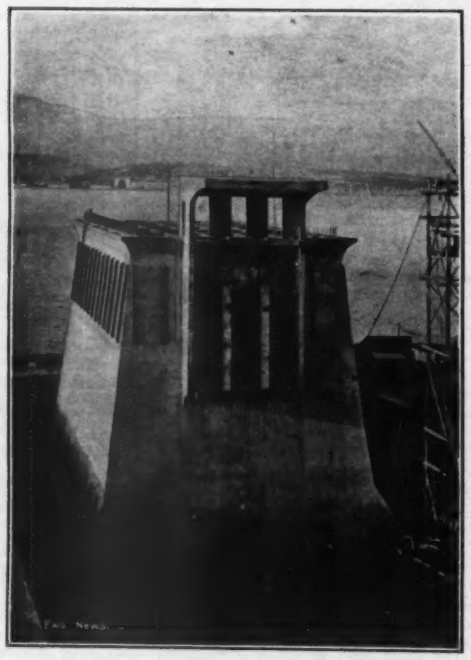


Fig. 1. The Caisson in Course of Construction on Shore.

a new torpedo of home design which at the same time could be supplied from their works in larger quantities than could be turned out at the Arsenal in case of war. As the Minister of the Marine accepted their offer, the company found it necessary to erect a testing station for the new torpedoes. A site along the coast at 20 miles from Toulon was chosen for the purpose, in the Bay of Leoube, a part of the roadstead of d'Hyères.

The bay in which the station is located has a depth of 30 to 40 feet, and a sandy, rockless bottom. The station is practically an artificial island situated in the middle of the bay so that there is a clear distance of from three to four miles in front of the torpedo tubes, into which the torpedoes to be tested may be sent. The rear of the station is about 800 feet from the shore. As will be observed, from the figures, the general form of the structure is that of a truncated pyramid. The main body is built of reinforced concrete, leaving a large chamber in the central part for the maneuver of the torpedoes, while upon the top of the concrete foundation block is built a superstructure containing a number of chambers such as are needed for various purposes in the testing of torpedoes. The superstructure was built upon the foundation block after the latter had been put in place upon the sea bottom.

*Abstracted largely from paper by Mr. M. Michel-Schmidt, in "Mémoires et Travaux de la Société des Ingénieurs Civils de France," Bulletin, Jan., 1909, p. 118.

The main block was constructed in a dock at the port of Hyères, and was then towed for a distance of 20 miles to its destination at Leoube, when it was let down upon a prepared foundation bed on the sea bottom at a depth of 39.5 feet. Here the sea bottom had been dredged off down to solid rock, making about 15 feet extra depth, and the foundation platform for the structure was made by filling up with small rock to 11.5 feet height or at a level of about 39.5 feet below the water line. The surface of the rock foundation was carefully leveled off by divers in order that the structure could be placed upon it under the best conditions.

The reinforced-concrete structure (Figs. 2-3) is 51 ft. in height* and at the base it measures 77 ft. long by 48.5 ft width. A flanged projected portion or sole at the base, lying along the sides of the mass, increases the width to 55 ft. At the top, the structure measures 70 ft. length by 36.5 ft. width. The total height of 51 ft. comprises a height of 39.5 ft. below water level and 11.5 ft. above the surface. The height is exclusive of the superstructure which is erected upon the top of the concrete block.

Referring to the sectional views (Figs. 2-3), the block is made up of comparatively thin walls built of reinforced concrete, together with a base of the same, and the interior is divided into a number of compartments by a series of vertical partition walls of reinforced concrete. The compartments are filled up either with a lean concrete or with sand. The outer walls are 6 ins. thick and are reinforced by a series of short horizontal webs. There is a camber in the transverse section of the base, 2 ins. in the central part, in order to avoid a perfectly flat surface and to give greater facility for anchoring the mass upon the foundation bed. A thickness of 6 ins. is used for the vertical partitions. The entire bottom of the structure is filled in with lean concrete up to a height of 6.5 ft., using a lime concrete containing 338 lbs. of lime per cubic yard total. The outer compartments lying next the side walls are then completely filled up with a lean cement concrete, this containing 420 lbs. of cement per cubic yard total. In the compartments which come immediately next the former in the interior of the structure, there is given a piling of lime concrete using 150 lbs. of lime per cubic yard. The rest of the compartments are filled up with sand to near the level of the flooring of the central chamber, and upon the sand is placed a covering of lime concrete of the quality last mentioned.

The chamber which is left in the central part of the block measures 48 ft. long and 19.5 ft. wide, and is used for the torpedo launching. This chamber lies in part below the water line, and near the base of the chamber are three torpedo tubes for launching under water. In the upper part of the chamber and lying above the water level, there are two torpedo tubes which are used for launching torpedoes above the surface of the water. These latter tubes are mounted in sockets of reinforced concrete, with their axes lying at 9.7 ft. above water level, and they are closed on the outer side by watertight metallic shutters. The tubes in the portion lying below water level are contained in bronze sockets, with the axes at 9.7 ft. below the water line. The central tube lies in the axis of the main structure, while the other tubes are spaced 6.6 ft. on either side of the same. In order to give access to the outer part of the bronze sockets there are provided two sets of gates which can be let down in front of them. These gates are of oak and each set is made to slide in grooves which are placed on the sides, as the main construction projects out on either side for this purpose, thus placing the front wall with ends of the sockets in a recess. The latter can thus be shut off by means of the gates and the space can be readily pumped out when it is desired to reach this part of the work. In the upper part of the chamber is a 2-ton crane which is used for handling the torpedoes. On the

*Note that in the figures the metric system is used in all dimensioning.—Ed.

lower floor there are three sets of rails corresponding to the three launching tubes, so that the torpedoes can be handled upon small trucks.

Upon the main block of the structure which has just been described is built the superstructure, most of this latter having been constructed after the main block had been anchored upon the sea-bed foundation. The superstructure contains a number of different chambers such as are needed for the maneuver of the torpedoes, lodgings for the personnel, shops, etc. In the front part is the observation station, which is built upon a platform of reinforced concrete. This platform projects out over the end of the main block, and from this point there is a wide view given by large windows at the front part, which is in the form of a balcony. As the flooring of the balcony is covered with glass tiling, the launching of the torpedoes can be observed more easily. The roof of the whole structure is arranged as a terrace and is surrounded by a railing. From this terrace there is given a view over the whole surrounding region. At the front part is a pilot tower built of reinforced concrete which carries a signal lantern and mast. In the front part of the superstructure are lodgings or offices, while in the rear are two machine shops constructed in two stories, one above the other. In the center is the main hall for the maneuver of the torpedoes, and at either side of the same there is a large opening which overlooks the water. The opening on the north side is 10 × 30 ft., and on the south side, 8 × 22 ft. For the former is used a sail-cloth covering, but as the south side receives the shock of the waves, this opening is closed by a strong wood cover reinforced with vertical steel bars. Across the central chamber is mounted a steel beam of 61 ft. length upon which runs an electric trailing carriage of three tons. The beam projects out over the water and allows the torpedoes and other material to be shipped on board. The torpedoes can thus be transferred to the 2-ton crane already mentioned, so as to bring them to the launching tubes. All around the superstructure and on a level with the top of the main block, there is a narrow platform of 2.5 ft. width.

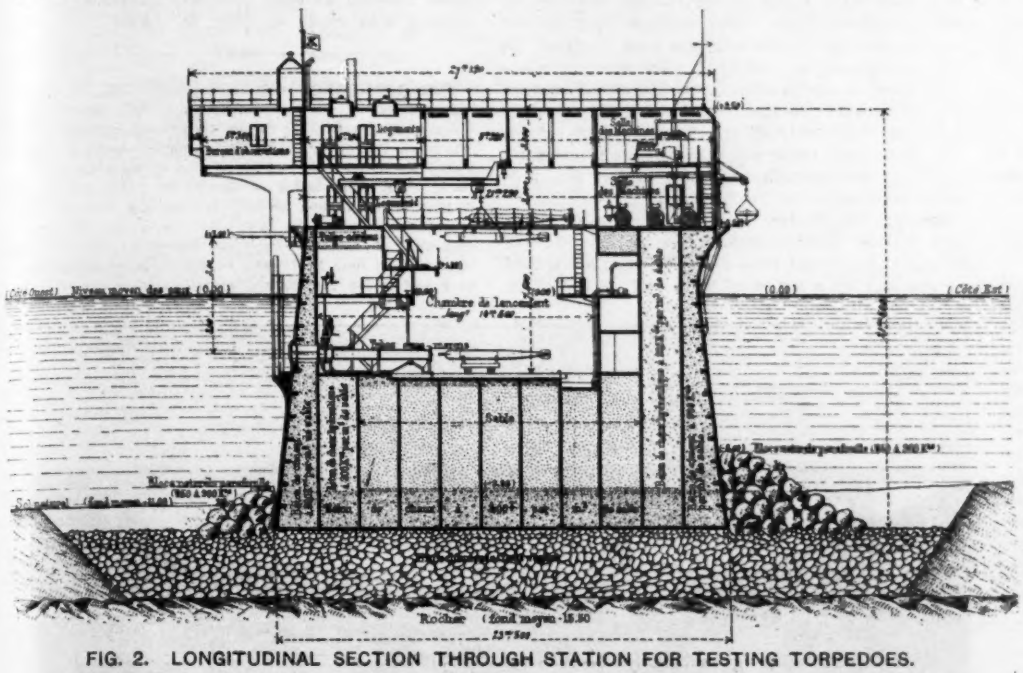


FIG. 2. LONGITUDINAL SECTION THROUGH STATION FOR TESTING TORPEDOES.

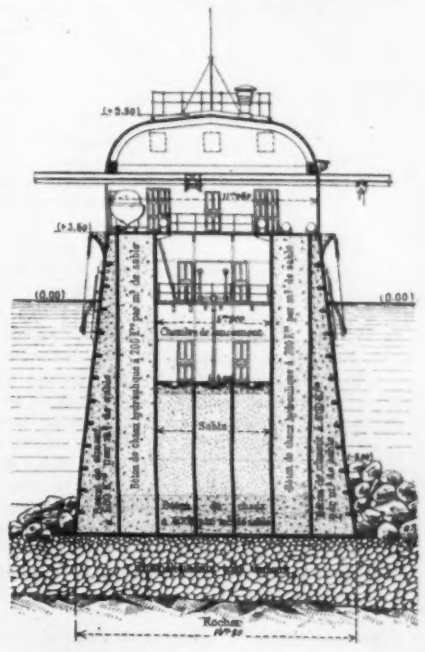


Fig. 3. Transverse Section.

Construction Details.

The construction was carried out by the Société Générale des Constructions en Béton Armé, otherwise known as the Dumesnil Company of Paris, in connection with the Hennebique firm. The actual work of construction was supervised on the spot by one of the general managers of the Dumesnil company, M. Trenaunay.

One of the docks belonging to the Government naval construction work at Toulon was used for the present purpose (Fig. 1). A good part of the work had to be carried out at the dry-dock, because in a reinforced-concrete structure it is more difficult to build on the additional parts after the main part has been towed to the spot than would be the case with a metallic caisson, where such parts can be readily bolted on to the main structure. For this reason it was found necessary to build the cross-planking of the top part as well as the main part of the front observation room overhanging the water. However, this construction was very unfavorable for the stability, so that special attention had to be given in order to secure this latter point, and the thicknesses of the walls in the upper parts were reduced to a minimum, as also the amount of cross-planking in the upper flooring. At the same time, in order to secure a sufficient stability, a part of the concrete was poured into the bottom of the structure.

In order to prevent the concrete from adhering to the flooring of the dry-dock, the surface of the dock was first covered with a strong packing paper, upon which was laid a cloth covering, so that the latter acted to absorb the excess of moisture. Then was commenced the building of the iron reinforcing part. This consists of a double web having a 6-in. mesh, built up of ½-in. round steel bar. Upon the bottom portion which was thus laid, there was erected the reinforcing for the vertical walls. The vertical irons were held in place by means of a scaffolding which covered the whole surface of the structure. In order to have the bottom floor of the mass quite watertight, a high-grade mortar was applied by the trowel all over the bottom before flowing on the concrete. For the composition of the concrete in the reinforced portions there was used Portland cement, sea gravel and sea sand. After the pouring of the concrete to form the bottom flooring of 6-in. thickness, concrete was applied to the outer walls, keeping the same thickness as for the floor, and using horizontal stiffening ribs outside. A set of removable wood panels was designed for molding this portion. Since the cross-section of each inner compartment was the same, the walls were built by using a series of wood frames of rectangular shape and of standard size. Such frames were disposed over the bottom of the work and the concrete poured into the intervening spaces so as to form the walls, in connection with the iron reinforcing web which had been erected in these portions.

The compartment walls were built up to a height of but 10 ft. in this way before the floating, and in order to give the proper stiffness it was necessary to build two series of cross-beams in reinforced concrete at the 26-ft. level and also across the top. In addition to this, the whole interior part was braced by a structure of cross timbers. At the top, a portion of the walls of the observation chamber was also built at this stage of operations. Since the upper part of the concrete block lies only 11.5 ft. above water level, some means had to be provided so that the caisson would not ship water from the action of the waves after it had been landed in place and during the construction work. Accordingly the upper wall is curved outwardly, as will be observed, which has the effect of rejecting the waves and overturning them backwards, thus protecting the work. On the western side this curved portion was made larger, so that the floor surface of the machine room could be very conveniently increased in this way. The next operation was to float the caisson in the dock, and this was rendered easier from the fact that the side next the water entry was lighter, so that it lifted first and thus the mass was gradually detached from the bottom. Two steam tugs were then attached by chains to tow the mass in the first place to arsenal basin No. 22. Here the walls of the outer compartments were finished up to 37.5 ft. height, while the low-grade concrete was filled in on the bottom to its standard depth of 6.6 ft. A pontoon lay alongside the caisson and served for the preparation of the concrete. The materials were brought from shore to the spot by two barges and a steam tug.

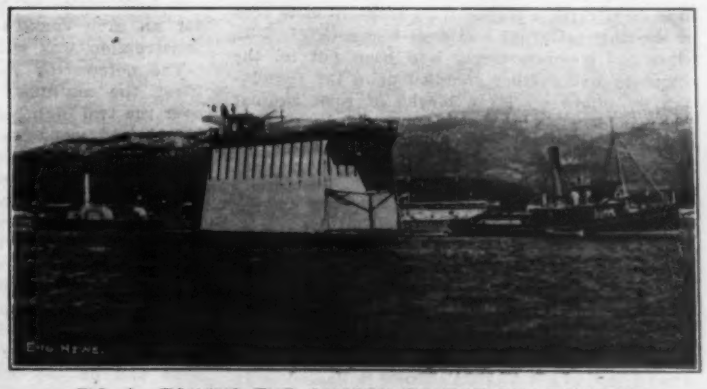


FIG. 4. TOWING THE CAISSON TO THE BAY D'HYERES.

After this part of the work was finished, the caisson was towed from Toulon to its destination at Léoube (Fig. 4). For protecting the surface, there had been placed around the upper part a series of oak timbers which received the towing cable. The whole upper surface of the structure was given a plank flooring so as to provide a platform for carrying out the landing operations. At this time the draft was 26.5 ft. and the displacement 2,800 tons. Two steam tugs were used for the towing operation. The distance from Toulon to the landing point is 19 miles in a straight line, but the actual trip was in this case about 22 miles. It required about 15 hours to cover the distance, starting at night in order to arrive at the landing place during the day and at an early hour. Although the operation was a difficult one, no special trouble was met with on this occasion, especially as there was fine weather prevailing.

After arriving at the site, the sinking of the caisson upon the foundation which had been prepared for it was next carried out, with the aid of four alinement masts planted at the corners by means of concrete supports, using cords stretched between the masts in order to show the place of sinking. Water ballast pumped from a steamer alongside was used to sink the caisson. Previous to this the interior had been made communicating and was divided, as regards the ballast, into three separate sections running longitudinally from end to end. By properly pumping in the water and at the same time maneuvering the cable attached to the caisson, the latter was sunk very regularly and arrived in place upon the bottom. An additional water ballast of 770 tons was then added for security.

The remainder of the work consisted in the filling up of the compartments and the building of the superstructure. The outer compartments were filled with concrete and the inner ones partially filled with sand. In the first place, the inner compartments received the sand filling and this gave sufficient ballast to allow of pumping out the outer compartments. These latter were then filled up with cement concrete. It was necessary that this latter work be carefully placed in the case of the outer compartments, since in case the wall of the cassion received a shock, the concrete compartments lying next the wall must be watertight. In carrying out this work the adjacent compartments were kept partly filled with water, so as to counteract the pressure of the concrete piling upon the partition walls. When the outer compartments had received the concrete filling, the adjacent range of chambers was pumped out, and then were filled up with low-grade concrete. In this way there is around the structure first a range of cells filled with cement concrete, next an inner range containing low-grade concrete, while the remainder of the inner cells are filled up with sand to a certain level. This disposition is seen in the two principal sectional views (Figs. 2-3). After the concrete filling had been put in, the structure was further steadied upon the foundation by piling up large blocks of rock against the floor sides. This was done at the latter part of the work in order to avoid giving shocks to the outer walls before the concrete filling had been put in place.

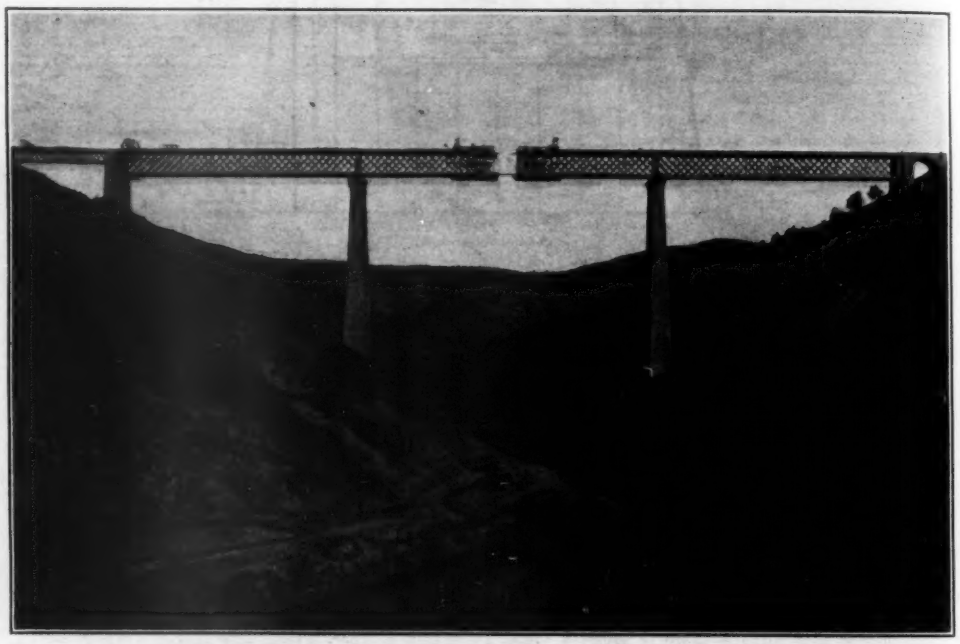
There remained to be constructed the portion which lies above the main platform of the caisson, or the superstructure mentioned above, and also the fitting up of the interior chamber, the adjustment of the torpedo tubes, and like operations. At the present time this work is nearing completion.

STATE MOUNTAIN RESERVATIONS in Massachusetts are being provided by the purchase of the highest mountain peaks. Mt. Greylock, Mt. Tom, Mt. Sugar Loaf and Deer Hill have been acquired by the state and it is quite certain that to these will soon be added Taconic Dome (Mt. Everett). The prime object of these reservations has been to afford recreation for large numbers of people. The first three of the reservations are already easily accessible by trolley. The reservations will also serve as game preserves, besides proving valuable as forest reservations. The "Springfield Republican" of June 20 gives an interesting description and historic account of these reservations.

Completion of the Steel Superstructure of the Fades Viaduct in France.

In our issue of June 8, 1905, p. 585, we described the design and a portion of the construction of the Fades Viaduct, which carries a railway across the Valley of the Sioule River, near the town of Vauriat in France. At that time the substructure had been completed and work was going on on the erection of the steel superstructure. For various reasons the erection of the steelwork has been delayed so that the joining of the final span was not made until May of the present year. Some details of the construction in the past four years are published in "Le Genie Civil," May 29, 1909, p. 81, and from that issue we take the accompanying half-tone and enough text to complete the previous description of the viaduct.

Briefly, the Fades Viaduct consists of three long lattice Warren spans, the middle one 472 ft. and the two end ones each 378 ft., approached on one end by a masonry arch and on the other by a 92 ft. latticed girder span. As originally designed and as illustrated in our previous article the approach now made with the short lattice span had two masonry arches. At the start of the construction, however, it was found that the foundations of the piers were too insecure for an arch construction so the adopted truss construction was substituted.



THE FADES VIADUCT OVER THE SIOULE RIVER, FRANCE.
(View taken May 2, 1909, 16 days before the final connection of the two halves of the middle span. Middle span 472 ft. Clear height above the bed of the river, 435 ft.)

The noteworthy features of the viaduct are: First, the continuity of the three large trusses over the two high piers and, second, the extreme height and slenderness of the two middle masonry piers, which are some 303 ft. high above the surface. The level of the top of the rail is 435 ft. above the bed of the river.

In building the steel work the two end 378-ft. spans were first erected upon falsework and the middle 472-ft. then erected from each end as a cantilever meeting in the middle. As noted, these three large spans are to be continuous when completed, fixed at one end pier and movable on rollers on each of the two high middle piers and on the other end pier. The 378-ft. span on one side was erected resting on the abutment upon a nest of rollers and on the pier upon a fixed chair which was raised to an elevation to care for the 13-in. deflection figured to occur at the outer end of the extended half middle span. On the other side the 378-ft. span rested upon the pier on a supporting plate controlled by a battery of hydraulic jacks capable of moving the plate and the end of the bridge in any direction. On the abutment this span rested on a support controlled by a small battery of hydraulic jacks which could move that end of the span transversely, pivoting it about the pier. Thus the fixed span was carried to the middle where it was approximately in the correct position and the correct alinement and elevation cared for at the juncture by the movement of the other span through its two sets of jacks. This joint was bolted up, the jacks removed and the proper roller or fixed bearings then inserted at each pier and abutment. The joining was made on May 18, 1909.

THE EFFICIENCY OF THE BICYCLING MUSCLES was studied by Prof. T. G. Benedict and Prof. T. M. Carpenter with the aid of the respiration calorimeter at Wesleyan University. The work and its results are published in Bulletin 208 of the Office of Experiment Stations, U. S. Department of Agriculture. The conclusion is that in work such as bicycling the leg muscles have an efficiency of 20 to 22%; that is to say, for every heat-unit of external work performed by the muscles they radiate four heat-units in the form of waste heat, over and above the normal radiation from the body at rest. The experiments were made on an "ergometer," which was a bicycle-like machine whose rear wheel was replaced by an eddy-current brake, i. e., a disk of copper running between the poles of an electromagnet. With fixed exciting current, this machine consumed a fixed amount of work per revolution. Its energy consumption was measured by placing it in the calorimeter, driving it from the outside by a shaft, and noting the heat absorbed by the calorimeter; this amount divided by the number of revolutions gave the energy-consumption per revolution. Then, a man mounting the machine in the calorimeter and riding it for a period of several hours, the heat radiated by man and machine was measured, and the number of revolutions was recorded. Part of the radiation, of course, is normal, since the body radiates heat when at rest. Therefore a measurement of the normal rest radiation was made (in the calorimeter). This being deducted from the total radiation during the work experiment left as remainder the amount of heat which the body radiated because of the muscular work it was doing, plus the radiation from the ergometer due to the work delivered to it. The ergometer radiation being known in terms of revolutions and exciting current, its value followed directly from the preliminary test-run. The ratio of the total net radiation (i. e. less the radiation of body at rest) to the ergometer radiation was in all cases very close to 5:1, meaning that the body radiated four heat-units over and above its normal radiation, for one heat-unit that it actually delivered to the ergometer through the leg-muscles. It was found that the ratio was practically the same in all trials, although in some of them the rider worked much harder than in others, and although one of the riders was a professional bicyclist while others were new to the bicycle. In other words, the efficiency of the muscles in question is independent of the rate of working, and is no higher for a professional of long experience than for a beginner.

Method of Keeping Data Relating to Street Lines and Grades.

By H. A. VARNEY.*

The necessity for keeping a careful record of all street-line work is appreciated by every municipal engineer and the following method used by the Engineering Department of the Town of Brookline may be of interest.

All records are kept on sheets 5 ins. × 7¾ ins. in size, ruled and printed in such a manner as to provide suitable space for all necessary data. The sheets are filed away vertically in drawers, under the street name, thus doing away with the necessity of any other index system. Long streets are subdivided by guide cards under cross-street names, so that any required sheet is easily found, regardless of the amount of data there may be relating to any one street. The sheets are punched so they can be carried in loose-leaf books, and each head of party is provided with a cover and is expected to keep in it all sheets which he takes from the files.

The upper portion of the "Street Data" sheets (Fig. 1) is reserved for the sketch of the street, which is usually drawn to a scale of 100 ft. to an inch. On the sketch are indicated the radii of the curves, the stations of the P. C.'s and of intersecting streets. The lower part of the sheet is ruled with columns for the stations, chord distances (left, center and right), deflections and grades (curb and side line).

Streets of the ordinary width are stationed on the center line, but on streets that are over 100 ft. wide the sides are worked up separately, and the stationing in such cases is done on the side lines with chord distances figured for 2-ft. offset and curb lines. These chord distances are all proportional so that the same deflection angle applies to the three chords. With these data one may start at any given station on either side or center and run forward or backward.

Stations are always figured on the arc, but "chord distances" take account of of any difference between the arc and chord of more than .005 ft. It might seem, at first thought, to mean a great deal of unnecessary work to figure the whole length of a street in this manner, but when one considers the number of times line and grade have to be given on a street for grading the abutting property, for sidewalks, curbs, gutters, resurfacing the roadway, etc., it will be seen that it is no waste of time.

There are several advantages in this method:

1. As the notes are worked up as soon as the plan is made for the acceptance of the street, it is an additional check on the accuracy of the plan.

2. As these notes are carefully checked, there is no danger of a party going into the field with incorrect data, and this eliminates a great source of error.

3. A whole party is not delayed while data is being looked up, for it is only a matter of a moment to find the sheet or sheets required.

4. It obviates the necessity of carrying valuable plans or note books into the field, where there is always the possibility of their being injured or lost.

5. Only the data actually required is taken from the office, another advantage over the ordinary note book which may contain information needed by some other party at the same time.

6. The data sheets are of use in many different ways in office work.

To keep a record of the grades given, another sheet (Fig. 2) is used, of the same size as the one above described and punched in the same manner. At the top are spaces for the street name and number, line and grade given, for what purpose, date, and names of the men in the party. Below these spaces are ruled vertical columns headed, Stations, +S., H.I., —S, Elev., Grade, Cut and Fill, in the order named, so the head of party has no excuse for not making a complete record of all work done. These sheets are filed away in a manner similar to the street data sheets, under the street and number. No more space is required for filing these sheets than for note books, and they are very much more accessible.

The accompanying cuts show the two forms of sheets: the first with sketch and data complete, and the second a record of line and grade given on the same street. This system has been in use in the Engineering Department of the town of Brookline several years and has proved very satisfactory.

*First Assistant Engineer, Brookline Engineering Department, Brookline, Mass.

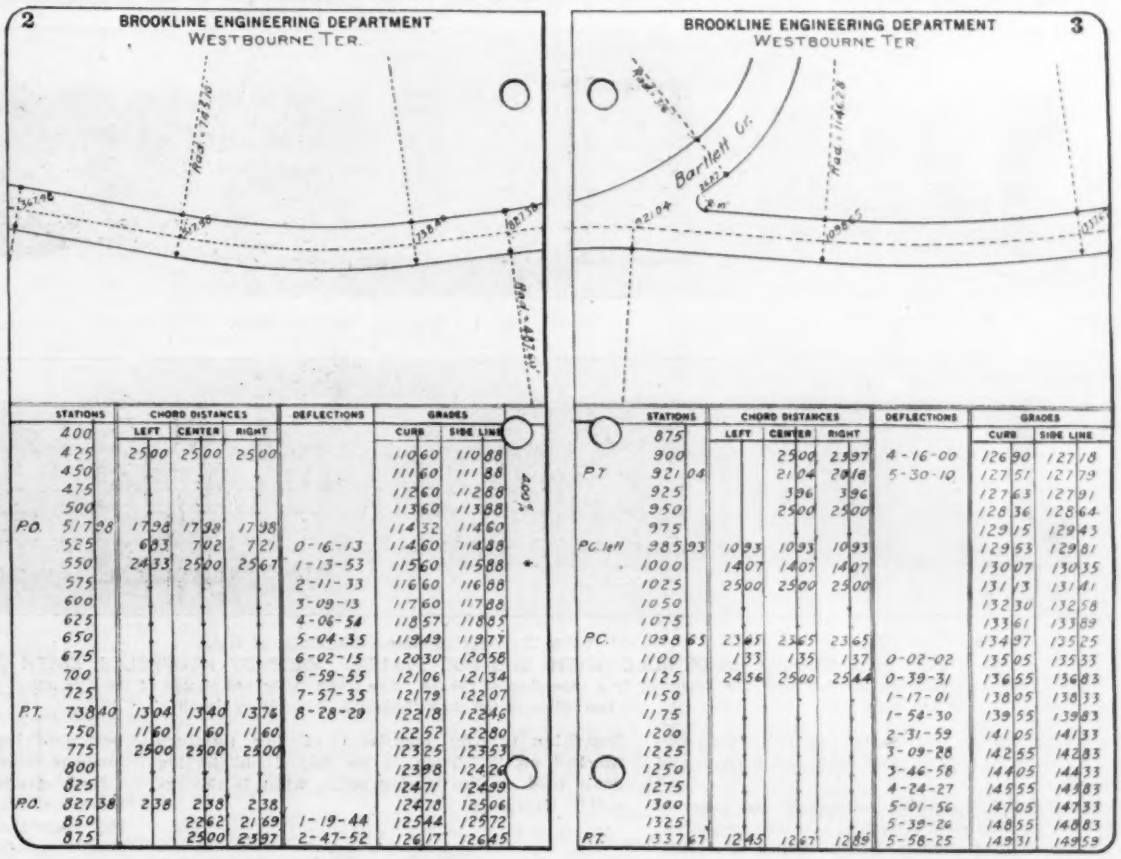


FIG. 1. STREET DATA SHEETS USED BY THE ENGINEERING DEPARTMENT OF BROOKLINE, MASS.

(Actual size of sheets is 5 ins. × 7¾ ins. The street sketch is made, on the original, to a scale of 1 in. = 100 ft.)

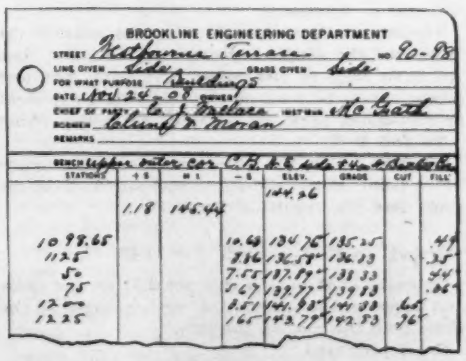


Fig. 2. Record Sheet for Street-Line and Grade Data, Brookline, Mass.

(Sheet shown is for same streets as those in Fig. 1.)

THE PRODUCTION OF COPPER IN 1908.—An advance statement prepared by the U. S. Geological Survey gives the total production of copper in the United States for 1908 as 942,570,721 lbs. This is the largest figure ever reached, exceeding that of 1906 by 24,765,039 lbs. and that of 1907 by 73,574,230 lbs. or 8.4%.

The production of refined new copper of domestic origin was 875,849,129 lbs., an increase of 11.6% over 1907. The production of refined copper was distributed as follows:

	Domestic origin.	Foreign origin.
Electrolytic	656,179,349	218,716,268
Lake	195,503,099	....
Casting	24,166,681	134,726
Total	875,849,129	218,850,994
Total output, domestic refineries	1,094,700,123	

Returns from all smelting and refining companies showed the following stocks on hand:

Jan. 1, 1908	125,745,796
Jan. 1, 1909	121,876,759
Stocks decreased during 1908	3,869,037

Undelivered sales and stocks carried by consumers and brokers have been excluded. In addition to the stocks of refined copper there were at smelters, in transit to the refineries, and at the refineries blister copper and material in process of refining to the amount of 175,254,659 lbs. Jan. 1, 1908, and 234,013,843 lbs. Jan. 1, 1909.

The following table shows the estimated annual consumption:

	1908.	1907.
Total refinery output	1,094,700,123	1,032,516,247
Stock at beginning of year.	125,745,796	46,497,181
Total available supply	1,220,445,919	1,079,013,428
Refined copper exported	*618,613,842	465,496,007
Stock at end of year	121,876,759	125,745,796
Total withdrawn from supply	740,490,601	591,241,803
Apparent consumption	479,955,318	487,771,625

*Exports of pigs, bars, ingots, plates, etc., reported by the Bureau of Statistics, less 43,262,285 lbs. of blister exported without refining.

In addition to the new copper consumed probably 23,000,000 lbs. derived from secondary sources, entered into the year's consumption.

A Duplex Slide-Rule with Log-Log Scales for Power and Root Calculations.

Operations with powers and roots of numbers are the most troublesome and awkward that the Mannheim slide-rule is required to handle. To solve exponential equations with fractional exponents particularly, the "logarithm scale" or scale of equal parts on the back of the slide must be drawn into service, which, in practice, proves to be cumbersome and liable to error. So simple a calculation as the following:

$$5.22^{1.46}$$

performed on the Mannheim rule is sufficiently awkward and liable to error that most people will use tables rather than the rule. The case is little better with integral powers and roots, excepting always the square, and for some problems the cube, fourth, sixth and eighth roots and powers.

Exponential equations nowadays are common enough to create a need for a more convenient instrument for solving them. Thus, hydraulic formulas of the exponential type have been current for a long time past. Numerous calculations in electric work, in steam, and in strength of materials involve exponential expressions or else contain hyperbolic logarithms. A very striking recent example in a different field is found in Taylor's formulas on metal-working, as set forth in his "The Art of Cutting Metals;" one of them is:

$$V = \frac{\text{Constant}}{F^{0.5225}\left(\frac{48}{11}D\right)^{0.3323+\frac{8.8}{66+48D}}}$$

which gives the cutting speed in feet per minute for a standard 1-in. roundnosed tool on a steel forging in a lathe for any feed F and any depth of cut D in inches. Special slide-rules have been constructed for solving this formula, but when they are not at hand logarithmic calculation is necessary, which is bound to prove very tedious.

In short, there is a well-defined field for an instrument which will simplify exponential calculations without being restricted to a special formula, and possess at the same time as wide a field as the ordinary Mannheim slide-rule.

The principle of making a slide-rule for exponential operations has been known a long time, and slide-rules made accordingly, called log-log slide-rules, are not a new contrivance. They have not come into wide use, however. A rule of this type brought out recently by the Keuffel & Esser Co. appears to be a specially useful adaptation of the log-log principle. Beside the log-log scales it carries all the scales of the ordinary Mannheim rule, in what we believe is a rather more convenient form. Thus the rule is a complete substitute for the Mannheim, with a greatly enlarged scope. Its only disadvantage is the rather high price (the makers quote the rule at $8).

THE LOG-LOG PRINCIPLE.—The ordinary slide-rule graduations are laid off as logarithms of the numbers marked on the graduation. Two identical graduations of this kind may be slid along each other, giving the sum or difference of logarithmic lengths, so that product or quotient is read off on the numbers affixed to the graduation. Now, retaining one of the graduations, let us change the other by laying off distance corresponding not to the logarithm, but to the logarithm of the logarithm of the numbers marked on it. Then, if we add 2 on the log scale to 4 on the log-log scale, what is the result? Evidently, it is

Distance 4 on log-log + distance 2 on log scale
or, log-log 4 + log 2

which gives point x, on the log-log scale. Thus,

$$\log\text{-}\log x = \log\text{-}\log 4 + \log 2$$
$$\text{or,} \quad \log x = 2 \times \log 4$$
$$\text{or,} \quad x = 4^2 = 16$$

That is, we have raised 4 to the power denoted by the setting of the log, scale in this instance 2. We can just as easily raise 4 or any number to a fractional power, or extract any root, whether integral or fractional.

Suppose the log-log scale is on the body of the rule; and the simple log scale on the slide; then we have only to remember that both root and power are to be read on the rule, while exponent or root-index must be read on the slide. Thus:

To find $4^{1.16}$.

Opposite 4 on the rule set the beginning or index-point of the slide, and opposite 1.16 on the slide read the result on the rule.

To find $\sqrt[2.51]{12.43} = 12.43^{\frac{1}{2.51}} = 12.43^{0.398}$

Opposite 12.43 on the rule set 2.51 on the slide, and opposite the index-point or beginning of the slide, read the root on the rule.

Or, if preferred,

Opposite 12.43 on the rule set the index-point or beginning of the slide, and opposite 0.398 on the slide read the root on the rule.

The principle being simple, there are yet some difficulties in constructing a rule on this principle, which are worth a brief explanation. These concern the linear scale to which the log-log graduation should be laid off, and the setting of its index-point or beginning.

SCALE OF THE GRADUATION.—In plotting a log graduation it is to be remembered that the base of the logarithms is not important, because logarithms to different bases have a constant ratio:

$$\log_a x = k \log_b x$$
$$\text{where } k = \log_a b$$

Thus it is immaterial whether the Briggs base 10 or the Naperian base 2.718 be used as the base of the logarithms plotted, since the two will be identical if plotted to different scales of appropriate ratio. In other words, only the linear scale is of importance. The log graduation and the adjoining log-log graduation must therefore be laid out to suitably corresponding linear scales. As the two graduations represent logarithms of different kinds of quantity (of numbers and logarithms, respectively), the equivalence is to be determined as follows:

Both graduations must represent logarithms measured from a zero-point, i. e., a point where the logarithm is zero. The beginning of the simple graduation therefore must be marked with the number whose logarithm is zero, or the number 1. The log-log graduation, similarly, must begin with x, where $\log\text{-}\log_b x = 0$; we find that $\log_b x = 1$, or $x = b$, the base of the logarithmic values represented.

Further, the end-points of the two graduations must correspond. As the log graduation extends to the number 10 (in the case of a C graduation), its length is log 10. This same linear scale is to be applied to the log-log graduation, and its end-point will therefore be the number defined by the equality:

$$\log \log_b y = \log 10$$
$$\text{or,} \quad \log_b y = 10$$
$$\text{or,} \quad x = b^{10}$$

Thus we determine that the log-log graduation must extend from b to b^{10} on the length of the rule, if it is to be used with a C graduation, i e., one extending from 1 to 10.

CHOICE OF BASE; POSITION AND RANGE OF SCALE.—The base b of the logarithmic values represented on the log-log graduation may be any desired quantity, without affecting the validity of the preceding deductions. But the range of numbers which the graduation represents depends directly on the base, since we saw that graduation will extend from b to b^{10}. Thus, if we use 10 as the base, the range of the graduation is

$$10^1 \text{ to } 10^{10}$$
$$\text{or,} \quad 10 \text{ to } 10{,}000{,}000{,}000.$$

This would be a fearfully crowded scale, and is needlessly extensive. On the other hand, using $e = 2.718$ as base, the range is

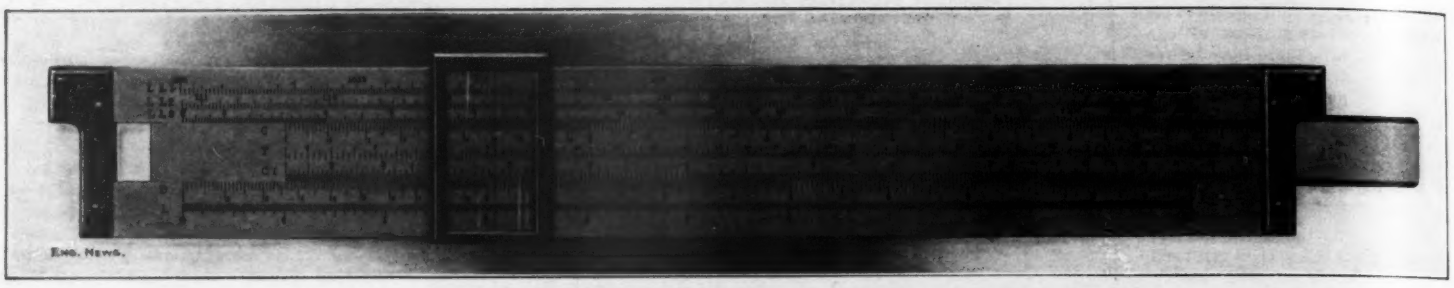


Fig. 1. Log-Log Face of Rule.

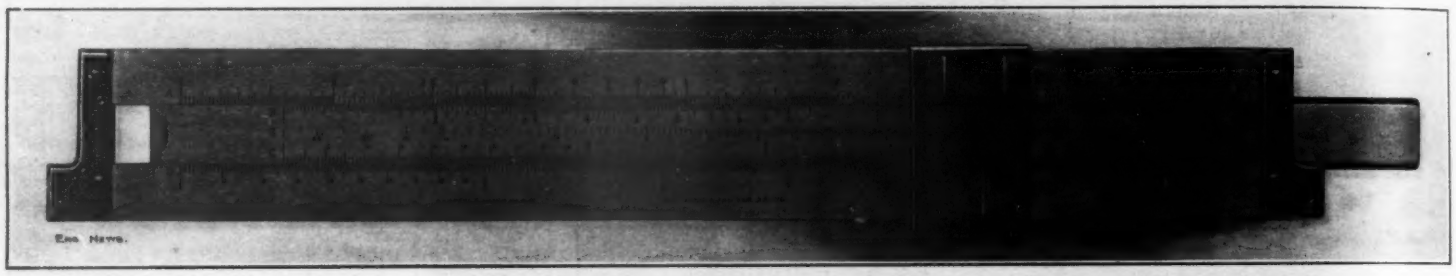


Fig. 2. Rear or Mannheim Face of Rule.

TEN-INCH DUPLEX SLIDE-RULE WITH LOG-LOG SCALES. MADE BY KEUFFEL & ESSER CO., HOBOKEN, N. J.

(The runner frames on the two faces are connected by a yoke-plate at the top. The screw in the end clamps of the rule allows adjusting the relative position of the two halves of the stock to tighten up or loosen the slide.)

e^1 to e^{10}
= 2.718 to 22,000

which is much better. In the rule before us, e is taken as the base.

Unlike the ordinary slide-rule graduations, these may not be read with arbitrary placing of the decimal point, since for example $6.2^{2.24}$ and $62.0^{2.24}$ are very different looking numbers. Therefore so long as the graduation of the slide (the log scale) represents numbers from 1 to 10, the log-log scale represents only the range of numbers just noted. But the log graduation also extends backward (to the left); successive rule-lengths give the numbers 0.1 to 1.0, 0.01 to 0.1, etc. Corresponding to each of these is a new and different rule-length of the log-log graduation with ranges $b^{1/b}$ to b^1 and b^{1/b^2} to $b^{1/b}$ respectively, or in our case, where e is the base,

$e^{1/e}$ to e and e^{1/e^2} to $e^{1/e}$

or approximately 1.105 to 2.718, and 1.010 to 1.105.

All these numbers may occur as roots or powers, which the rule ought to be able to deal with. On the other hand, the upward extension beyond 22,000, corresponding to the log scale from 10 to 100, would practically never be needed.

The rule here shown bears all three of the scales first mentioned, one directly above the other, so that by the help of the runner, any one of them may be used. In each case, however, it must be remembered that they correspond to different rule-lengths, and that when the point 2 on the slide is used to represent the exponent 0.2, the result is to be read on the log-log scale preceding the one on which the first setting is made; and likewise when graduation 2 on the slide represents the exponent 20, the final reading will be found on the log-log scale following or having higher numbers than the one on which the original setting was made. In other words, we must subtract or add an entire rule-length in the cases cited.

The selection of the logarithmic base (or the value b in laying out the graduation $A = \log_b N$) is really very unimportant, in spite of the apparent proof that e is preferable to 10 as base. For, it may be shown that the graduation comes out exactly the same whatever number be used as base, only that the starting-point differs. Thus, if we compare $\log_{10} \log_b N$ with $\log_{10} \log_e N$, we find that they differ by a constant quantity:

We know that logarithms to different bases have a constant *ratio*, so that

$$\log_e N = \log_e b \times \log_b N$$

Taking logarithms, we find that the log-logs have a constant *difference*:

$$\log_{10} \log_e N = \log_{10} \log_e b + \log_{10} \log_b N$$

or,
$$\log_{10} \log_e N = K + \log_{10} \log_b N$$

Thus, whether the log-log scale is based on e or on 10, it is exactly the same, only shifted either to right or to left by the amount $K = \log_{10} \log_e 10$, which, obviously, is the distance from the point e of the log-log scale (its starting-point in our case) to graduation 10. Therefore, by setting the slide with its index-point at 10 of the log-log scale, we have exactly the same state of affairs as though the log-log scale had been based on 10 originally.

It is to be noted that the numbers on the log graduation are logarithms of the adjoining numbers on the log-log scale. For, at any distance x from the index-point, the two scales showing N and M, we have $\log N = x$, and $\log \log_b M = x$. Therefore $N = \log_b M$. Since the rule here shown is graduated with e as base, the slide in its zero position shows directly the hyperbolic logarithms of the adjoining points on the rule. By shifting the slide so that its index-point is on 10 of the log-log scale, Briggs logarithms are read off. By setting the slide to any other number, logarithms having that number as base may be read, though this latter has no practical value, of course.

Thus the rule described enables us to read logarithms, either Briggs or hyperbolic, for the entire range 1.01 to 22,000. Also, any desired operations of involution or evolution may be performed, over the same range, with any exponent (within practical limits).

Proper fractions, i. e., quantities less than 1.0, cannot be handled directly on the rule, since they cannot be represented on the log-log scale, as even the number 1.0 is at infinite distance ($\log 1 = 0$ and $\log \log 1$ = negative infinity). But the reciprocal of a fraction may be dealt with instead, and the indicated operation performed upon it; then the reciprocal of the result will be the desired root or power of the fraction. Thus the rule covers the entire range of natural numbers up to 22,000, with the exception of the interval between 0.99 and 1.01. Quantities in this interval may be brought within range of the rule by multiplying them by a factor sufficient to make them greater than 1.01. Performing the desired operation upon both the new quantity and the factor and dividing the first result by the second, will give the root or power sought.

As to the general construction of the rule, and its graduations other than the log-log scales, we may refer to the accompanying views of front and back of the rule.

CONSTRUCTION OF RULE.—The rule is graduated on a length of about 10 ins. It is considerably wider than the ordinary slide-rule, being 1⅝-in. wide, to accommodate the increased number of graduations. The stock is in two separate pieces joined by German-silver end clamps, and the slide is of equal thickness with the stock, so that both sides can be utilized. Both front and rear are faced with celluloid.

The ends of the rule project fully 1¼ ins. beyond the graduation, not only to accommodate the clamps and allow the runner to reach the end of the graduation, but also to furnish a handle for manipulating the instrument. The slide projects ¼-in. beyond the clamp at either end, to give a good end hold in every position of the slide.

The end clamps are fastened to the upper strip of the stock by two rivets, and to the lower strip by a screw, these fastenings in each case passing through from the front to the rear plate. The holes in the wood for the screws are slotted transversely to the rule, so that the two strips of the stock may be set up tighter against the slide whenever necessary, although the two-piece construction tends to maintain the original fit of the slide much better than the solid stock of the ordinary form of slide-rule, in which any warping of the thin connecting web affects the fit of the slide.

The runner consists of two glazed frames, one on each face, connected by a yoke plate at the top edge of the rule. The frames have claws extending over the bottom edge of the rule, which are held to a bearing by a spring under the yoke-plate. The hair-lines on the glass of the two frames are set to coincidence.

GRADUATIONS.—On the front face, the place of the usual *A* and *B* scales is taken by the log-log scale and its corresponding simple log scale, the latter being identical with the ordinary *C* scale, i. e., graduated from 1 to 10 in the length of the rule.

The place of the usual *C* scale is occupied by an inverted *C* scale (i. e., a *C* scale graduated from right to left), marked *C I* and numbered in red, to distinguish it clearly. The *C I* scale facilitates obtaining reciprocals, since reciprocals can be read off directly without shifting the slide. Adjoining it is the ordinary *D* scale on the lower part of the rule. Two auxiliary graduations also appear on this face: a Tangent scale along the middle of the slide, to be used in conjunction with the upper *C* scale (location *B*); and a Log scale or scale of equal parts below the *D* scale, from which may be read the common logs of the numbers found on the *D* scale. This may be used for checking Briggs logarithms read from the log-log scale, and on part of the range it gives them more accurately than the latter.

The rear face has *A*, *B*, *C*, and *D* graduations identical with those of the ordinary Mannheim slide-rule. A Sine scale extends along the middle of the slide.

FUNCTIONS.—In so far as the rear face and the three auxiliary scales are concerned, the calculations possible with this rule are the same as those performed on the Mannheim rule. The Sine scale reads degrees and subdivisions from 0° 40′ (sin = 0.01) to 90° (sin = 1.0), corresponding values of natural sine being read on the *B* scale. The Tangent scale is graduated in degrees and parts from 5° 45′ (tan = .1007) to 45° (tan = 1.0), and values of the natural tangent are read on the *C* scale. But operations with the tangents and cotangents are easier on this rule than on the ordinary rule because there is also a *C I* scale adjoining, so that cotangents from 0° to 45° and tangents from 45° to 90° are read directly instead of by setting the slide. Further, all calculations involving trigonometric quantities are made easier by the fact that the graduations are on the face of the rule and can be used in the simplest way in direct conjunction with the regular scales.

The log-log group of scales has already been explained in principle, but not in a way to exhibit the range of calculations to which this part of the rule is adapted. The following suggestions cover the most interesting calculations.

Exponential Calculations.—Extracting any root or raising to any power, integral or fractional, is typified by such expressions as were given earlier. These are solved directly, reading root and power on the log-log scale and index or exponent on the log scale.

When the root and power are given, but the exponent is unknown, locate both numbers on the log-log scale, set the index of the log scale to one of them, and under the other number find on the log scale the desired index.

Calculations Involving Natural Logs.—Formulas in which natural logs appear as multipliers are not rare. Directly below any number on the log-log scale, its natural log will be found on the upper scale of the slide (when slide is in zero position) and also on the *D* scale (slide may be in any position). Multiplication and division may then be performed with it on the *C I* and *D* scales, or exponential operations may be performed with the nat. log. as either root or exponent just as with any other numbers, by means of the log-log scales.

Damping Curves.—Damped vibrations are represented by formulas containing e^{-at}. This quantity is the reciprocal of e^{at}, and the latter is found on the log-log scale opposite values of at read on the slide in the zero position of the slide. Then the reciprocals of all the values e^{at} may be read off the *C I* scale opposite the values e^{at} on the *C* or *D* scales.

Catenary Curves.—All calculations with catenaries involve the fundamental expression

$$\text{Deflection} = \frac{1}{2a}\left(e^{\frac{ax}{2}} - e^{-\frac{ax}{2}}\right)^2$$

both abscissas and deflections being measured from the lowest point, or vertex, and a being the quotient of weight per foot divided by horizontal pull. Such expressions are easily handled by the rule, $e^{\frac{ax}{2}}$ being read on the log-log scale opposite values of $\frac{ax}{2}$ on the log scale, with slide in zero position. Then $e^{-\frac{ax}{2}}$ is the reciprocal of the quantity so found, and may be read off by use of the *C I* scale. The rest of the computation is self-evident.

Compound Interest.—A sum of money earning interest at the rate r, compounded annually, has after n years the value

$$C \times (1 + r)^n$$

The rule gives the results very quickly, by setting the index of the slide opposite point $1 + r$ on the rule, and reading $(1 + r)^n$ on the rule opposite n on the slide. The multiplication of the result by *C* is then done on the lower scales. The smallest value of rate of interest which the rule will handle directly is 1%, giving $1 + r = 1.01$. Smaller rates must be dealt with by multiplying, and later dividing by the corresponding power of the factor used, as already explained. The principle of the calculation is the same, of course, for semi-annual or quarterly compounding.

Such a problem as finding the time required

for a sum of money to double, or triple, or reach any other particular value, is just as simple. Thus, suppose we wish to know how long it takes for money at 4% compounding semi-annually to increase by 50%; to double.

The basal rate being 2% (per half year), the total amount is $U \times (1.02)^n$, where n is half years; and we wish to find the values of n which will make the coefficient become 1.50 and 2.00 respectively. Set index of slide opposite 1.02 on the rule, and under 1.5 and 2.0 on the rule find on the slide 20.45 and 35.0. These are half-years, and the periods therefore are 10 yrs., 2 mo., 21 d., and 17 yrs., 6 mo., respectively.

An "Accelerated Test" of Road Wear By Automobile Traffic.

Unusual circumstances subjected a short stretch of road in eastern Germany to very heavy traffic of automobile busses for a period of five months during the winter 1907-1908, and in that period some interesting studies of road wear were made. The traffic was much heavier, both in amount and in loading, than normally occurs even on heavily traveled main highways, and the results represent what might be called an accelerated test of road destruction by motor-car traffic. The matter has recently been reported in the "Zentralblatt der Bauverwaltung" (June 5, 1909). The showing made is rather alarming, but in reality it only puts into specific figures what has been more or less generally recognized as true.

The two groups of cross-sections shown herewith as Figs. 1 and 2 give typical illustrations of how rapidly the traffic cut through the hard surface of the highway. The circumstances of the case were as follows:

A railway tunnel between the towns of Mettlach and Ponten caved in on Nov. 27, 1907; this interrupted the line from Trier to Saarbrücken and necessitated the employment of auxiliary means of transportation to bridge the gap for passengers and baggage. An average of 10 passenger-trains each way per day had to be taken care of.

For a fortnight local vehicles, such as cabs, farm wagons, etc., were employed. On Dec. 6 two closed motor-cars holding 12 to 15 passengers each were obtained. A week later ten large motor-busses belonging to the Grosse Berliner Motor-Omnibus Co. were put into service on the portage and the two smaller busses were dismissed. These ten busses were in service till early in February, when four of them burned, after which the remaining six handled the service.

The tunnel was restored ready for traffic by May 1, 1908, making the period of road portage practically five months. During this time the motor-busses averaged 80 trips between Ponten and Mettlach, though in the Christmas weeks the number of trips was increased to as high as 140 per day.

The stretch of road in question, excluding the terminal portions which lay in paved streets, is slightly under two miles long. A short stretch near the middle is level, lying on the ridge through which the railroad tunnels, while the rest is steeply sloped down toward the two ends. The northerly slope averages 6.9% grade, the southerly slope 8%. The road had a broken-stone pavement, generally a kind of Telford, i. e., there was a base course of shingle or cobbles. The upper course consisted of quartzite, stated to be very suitable for road purposes. The age of the road surfacing varied, but the important feature is that the older parts had been kept in careful repair, and the entire two-mile length was in first-class condition.

Perceptible wear began when the large motor-busses came into service. A picking-up action was noticed in the tire tracks, and in a few days the road was covered with fragments of stone torn out of the surfacing. Further, although the gage of the rear wheels was larger than that of the front wheels, the concentration of wear from tracking soon produced the results shown in the sketches.

These large busses are described as follows: Weight, empty, 13,000 lbs., of which nearly 9,500 lbs. was on the rear axle; capacity, 25 passengers; weight loaded, about 17,000 lbs.; tires, solid rubber, width 4 ins. front, 8½ ins. rear; gage, 5.9. ft. front, 6.6 ft. rear; speed, 6 to 15 mi. per hr.

The rutting of the road once started, it developed in a short time so far as to form grooves up to 6 ins. deep by 12 ins. wide, and ridges formed alongside the ruts from the displaced material. The ruts were not clean but contained much loose material, which the following wheels either pushed aside or crushed.

Repair work was started as soon as the destructive actions were noticed, and was continued to the end of the period. Coarse broken stone was placed in the ruts and pressed down with a steam roller, whereupon a binding course of small stuff such as cinders and coarse sand was similarly applied. This could be done only when the road was not frozen.

Two weeks often sufficed to destroy the repair work completely. During the last three months it was a constant struggle to keep the road in passable condition. If heavy continued rains had occurred in March or April it would have been impossible to maintain the traffic.

The wear and grooving was worse in December. Freezing weather in January and the first part of February held matters stationary and preserved the road, though in badly rutted condition. Thereafter the southerly slope thawed first, and repair work was concentrated on it, the northerly slope being taken up later.

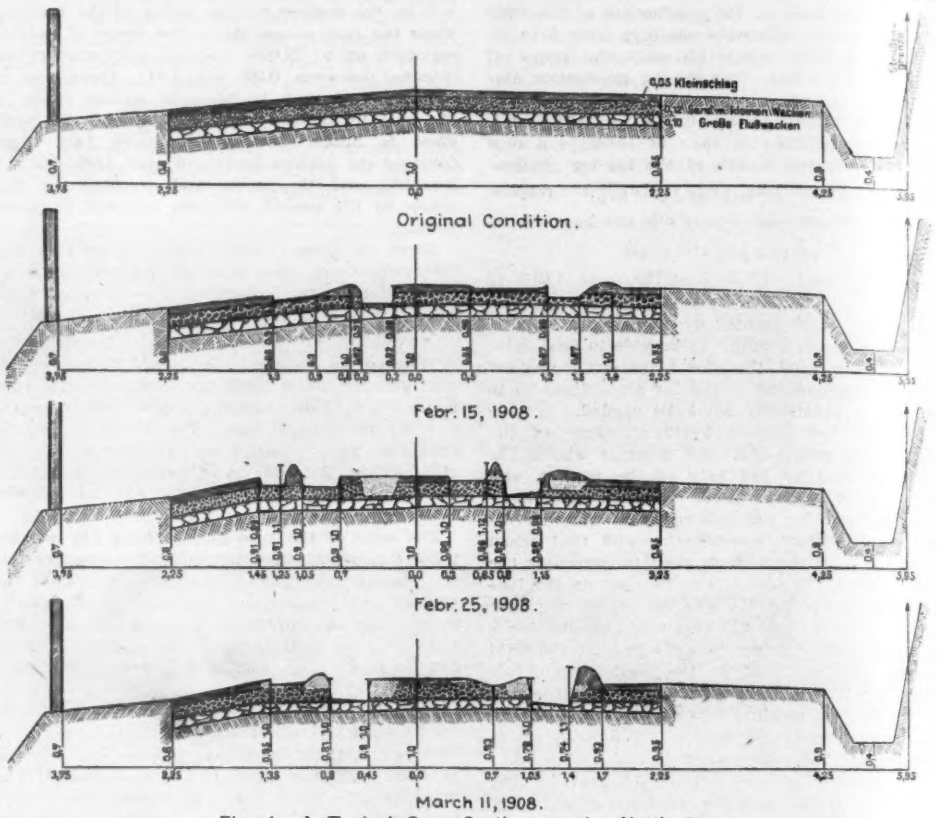


Fig. 1. A Typical Cross-Section on the North Slope.

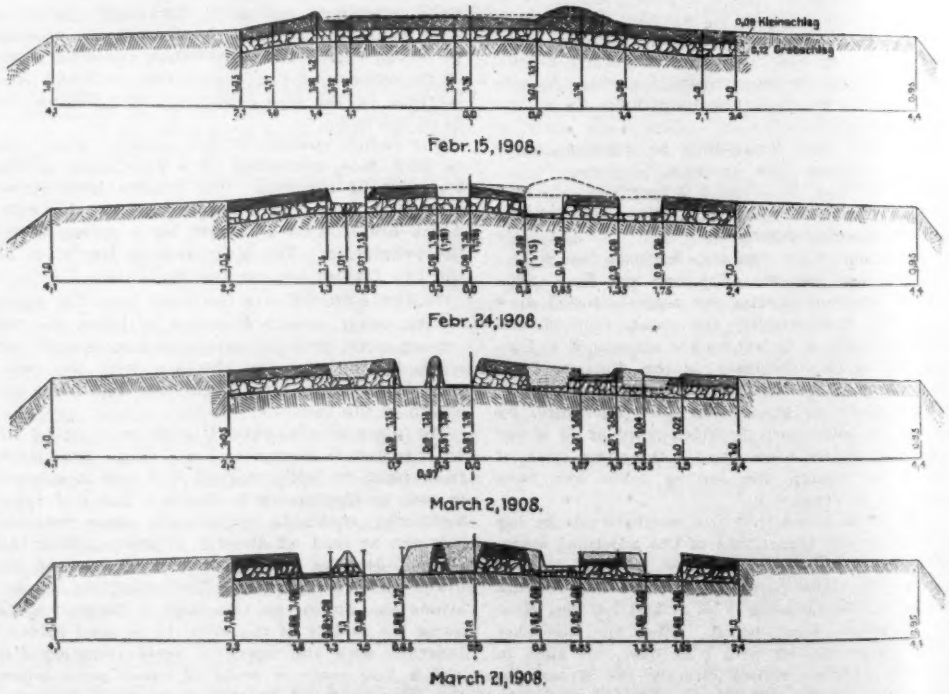


Fig. 2. A Cross-Section on the Upland.

FIGS. 1 AND 2. THE EFFECT OF CONCENTRATED MOTOR-BUS TRAFFIC ON A TELFORD-PAVED HIGHWAY IN GERMANY.

The five-months' maintenance cost about $4,000, or over $2,000 per mile. About 1,250 cu. yds. broken stone and an undetermined amount of sand and cinders were used. Mr. M. Gorz, who reports the details, says that but for good weather, a convenient supply of materials and the availability of labor from the Railway Department it would not have been possible to keep up the road. These favorable factors, we conclude, also operated to reduce the cost.

The influence of the heavy weight concentrations upon the destruction of the road was

Fig. 3. A View of the Rutted Road-Surface in Winter.

evident in one of the paved streets at one end of the route. A street newly paved with stone block, but apparently without concrete base, was used temporarily to detour around the main street. But in a few days the surface was deeply grooved, the wheels crushing the stone blocks down into the soil, and the busses had to use another street.

Garbage Reduction Plant at Vincennes, Ind.

In the earlier days of garbage reduction, it was thought that this system of treatment was applicable only to cities of considerable size. Within the last few years a number of quite small plants have been installed. One of these has just been built by the Vincennes Sanitary Co., of Vincennes, Ind., to treat the garbage of that city. The population of Vincennes by the census of 1900 was 10,249; presumably the place has increased materially in size since that date.

As will be seen by the accompanying illustrations, the Vincennes plant is remarkably compact. It has been so designed as to give scarcely any exposure of the garbage to the air from the time it is fed into the digestors until it comes out as a finished product. The plant was designed by Mr. F. G. Wiselogel, who has established several plants for treating garbage on the so-called Wiselogel system. The system is now controlled by the United States Construction & Utilization Co., of Rochester, N. Y. The Vincennes plant was installed under Mr. Wiselogel's direction.

The plant is housed in a reinforced - concrete building, 45 × 96 ft. in plan and 40 ft. in height. All the floors are of concrete construction and are provided with ready means for thorough washing and draining. The roof is of corrugated steel.

The plant consists of a single unit which, as will be seen by the illustrations, includes two digestors, a hydraulic press and a Eureka drier, all placed horizontally. The garbage is cooked by steam in the digestors for about two hours, and is kept in motion by revolving wheels meanwhile. When animals are treated, about four hours' cooking is required. The material from the digestors is discharged into the presses while boiling hot. The tankage from the drier goes by means of an elevator to the screening floor, and after screening it is taken to the shipping or storeroom. The whole process is said to occupy only five hours.

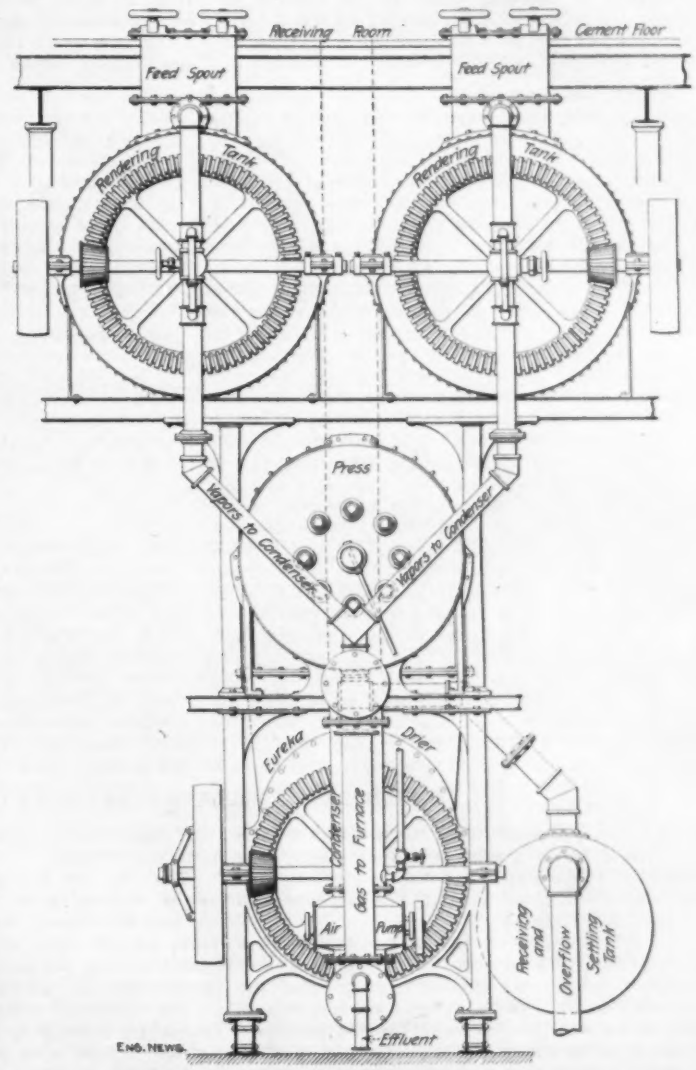


FIG. 2. FRONT VIEW OF STEAM DIGESTORS, HYDRAULIC PRESS AND EUREKA DRIER FOR THE GARBAGE REDUCTION PLANT, VINCENNES, IND.

The grease and water from the hydraulic press flow to a settling tank, where the grease is drawn off, after cooling, for shipment in either barrels or tanks. The water remaining after the grease is drawn off, together with the floor washings, is discharged into four septic tanks of concrete construction, located outside the building and between the building and the sewer. These tanks are ventilated by means of pipes, and provision is also made for ventilating the building. The vapors from the various receptacles which contain the garbage during treatment are passed through a condenser and from there to a cast-iron retort heated to redness, where they are burned.

The plant has a capacity of 20 tons of garbage and animal matter in 10 hours. Mr. Wiselogel informs us that three men are sufficient to operate a plant of this size and keep everything in good condition: a foreman, an engineer and a top-floor man.

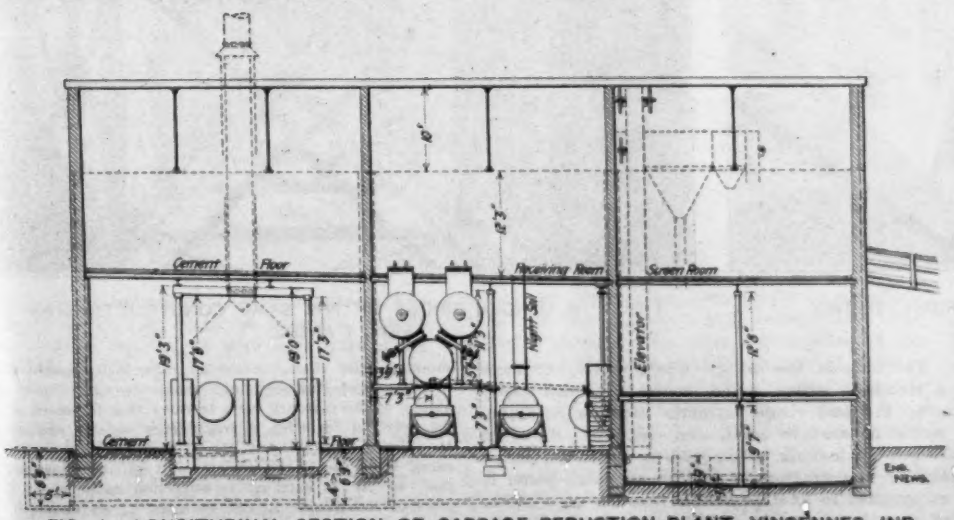


FIG. 1. LONGITUDINAL SECTION OF GARBAGE REDUCTION PLANT, VINCENNES, IND.

THE SUPERVISION OF WATER COMPANIES in Massachusetts has been put in charge of the State Board of Health by a legislative act approved April 26, 1900. Like so much other important Massachusetts legislation, the act in question is brief. We quote it in full as follows:

Sec. 1. Upon complaint in writing relative to the service furnished in any city or town, or the charges therefor, made by any company engaged in the business of supplying water to such city or town or to the inhabitants thereof, signed by the Mayor of the city or the Selectmen of the town, or by 50 customers of the company, and filed in the office of the State Board of Health, said board shall notify the company by leaving at its office or place of business in such city or town a copy of the complaint, and may thereupon, after notice, give a public hearing to the complainant or complainants and to the company, and shall require the company to furnish such information in its possession as may be necessary to determine the matters involved in the complaint, and after the hearing may make such recommendations concerning the reduction, modification or continuation of such charges for service, or concerning improvements in the quality of the service or extensions of the same, or concerning other matters in the premises, as the board shall deem just and proper. Any such recommendations shall be transmitted in writing by the board to the company complained of, and a report of the proceedings and of the result thereof shall be included in the annual report of the board, together with a statement of the action, if any, taken by the company upon the recommendation.

Sec. 2. This act shall take effect upon its passage.

A Curious Highway Bridge.

By S. E. FITCH,* Assoc. M. Am. Soc. C. E.

A unique highway bridge may be seen in the outskirts of Havana, Cuba, in the vicinity of Camp Columbia, recently vacated by the American troops. It is of a type known in Havana as "The American System," probably on account of it being a pin-connected structure rather than by reason of its design or manufacture. The fact that the depth and panel length are each 2½ meters indicates that it was not designed in the United States.

The bridge was originally of the double-intersection deck Warren type. It comprised two complete bridges set side by side to give the necessary width. The upper and lower chords as well as all web members and the end posts are eyebars, of sizes from 4 × ½-in. to 6 × 1 in. The accompanying Fig. 1 is a fairly complete sketch of the structure.

The compression members have, at intervals of about 19 ins., holes of ½-in. diameter drilled through them. It was evidently the intention of the designer to lattice or bolt the eyebars together in some manner in order to increase the lateral stiffness of the member. This, however, has never been done, except at the intersection of the web members, where a wooden packing block has been bolted between the eyebars. Vertical wooden posts have also been inserted in a crude manner at all panel points except at the ends of the bridge. The patching was doubtless done at some later date when members A B C D (Fig. 1) began to buckle. These members are now considerably sprung.

The pins are 4 ins. in diameter, and from all appearances extend through from one truss to the other, being covered by a wrought-iron pipe between trusses.

The top and bottom lateral system and the sway bracing are of 1-in. round bars which are adjustable at their intersections, where they pass into an 8-in. ring of 2 × ⅜-in. iron. Some of these braces, on being stressed, have distorted their rings into an elliptical shape.

The roadway consists of about 8 ins. of macadam resting on 2-in. longitudinal planking which in turn rests on transverse timbers 6 × 9 ins. by 12 ft. These timbers are spaced about 2 ft. c. to c., and rest directly on the eyebars of the upper chord, without being notched over them.

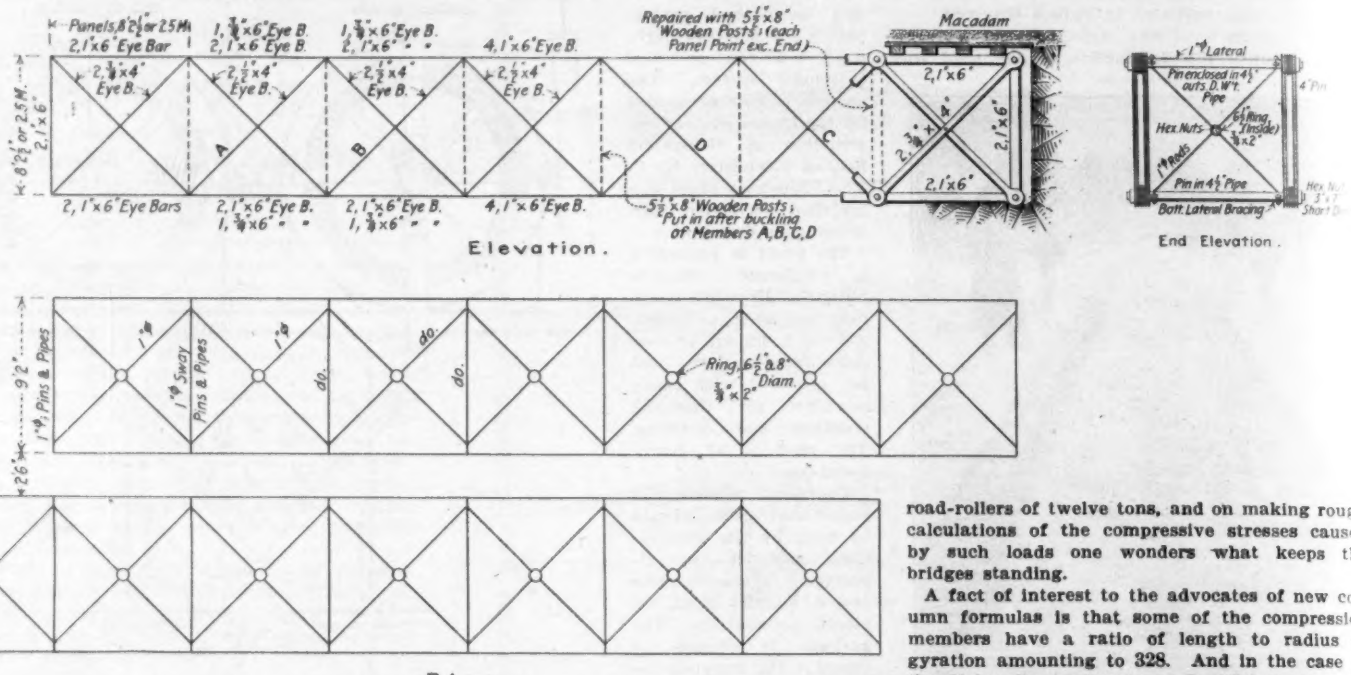


FIG. 1. SKETCHES OF A BRIDGE IN CUBA BUILT OF EYEBARS.

*Westfield, N. Y.

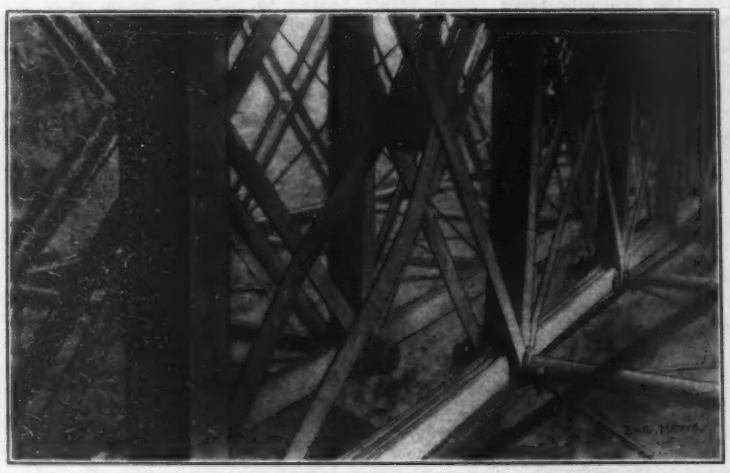


FIG. 2. BUCKLING OF EYEBARS IN THE BRIDGE SHOWN BY FIG. 1.

FIG. 3. A LONGER BRIDGE OF THE SAME CONSTRUCTION AS FIGS. 1 AND 2.

The idea of the design was probably to have a standard bridge, which could be hauled easily over the bad roads formerly existing in Cuba, would be easy to erect, and could be altered in length by leaving off or adding an extra panel. Indeed, the practice of adding an extra panel is exemplified in another bridge near by on a street of very heavy traffic. The latter bridge is the same in every respect as that shown by Figs. 1 and 2, except that it has nine panels in place of seven and has three complete bridges placed side by side to give the required width. This bridge, of which Fig. 3 gives a view, has now failed so badly in its compression members that the wooden patches are carrying most of the load. Both of these bridges have to carry steam road-rollers of twelve tons, and on making rough calculations of the compressive stresses caused by such loads one wonders what keeps the bridges standing.

A fact of interest to the advocates of new column formulas is that some of the compression members have a ratio of length to radius of gyration amounting to 328. And in the case of the ½-in. diagonals, some of which were compression members before the introduction of the vertical wooden posts, the length ratio is even greater.

ABOUT 200,000 FOREST SEEDLINGS were distributed from the Vermont State Nursery last spring. These seedlings went to every county in the state. The state nursery at Burlington, Vt., is being enlarged. It is expected that 1,000,000 trees a year will be demanded before long. Mr. A. F. Hawes has been appointed State Forester of Vermont, under recent state legislation creating that office. Mr. Hawes is also connected with the Agricultural Experiment Station at Burlington, Vt., as Forester.

THE SUFFICIENCY OF THE QUEBEC BRIDGE piers for the new bridge which is now being designed, has been questioned. The new design calls for a bridge much heavier than the old and the three members of the Board of Engineers are unable to agree upon the ability of the masonry piers to carry this increased load. As provided for such a contingency outside engineers, Mr. Henry Holgate and Mr. Alfred Noble, have been called in to decide the question. It will be remembered that, so far as external investigation could show, the piers of the Quebec Bridge were not at all damaged in the fall of the superstructure.

ENGINEERING NEWS

A Journal of Civil, Mechanical, Mining and Electrical Engineering

Published every Thursday by

THE ENGINEERING NEWS PUBLISHING CO.

220 BROADWAY, NEW YORK

BRANCH OFFICES

CHICAGO: 1636 Monadnock Block
WASHINGTON, D. C.: Home Life Building
SAN FRANCISCO: Monadnock Building
ATLANTA: Candler Building
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Well-meaning people now and then, and ill-meaning people perhaps more frequently, propose minority representation by a city on the directorate of a public-service corporation as all the public control that can be needed to safeguard the interests of the consumers and the city. Such a plan is generally highly disappointing to all save the representatives of the private corporation involved. This has very recently been shown to be the case at Philadelphia, where the city was given representation on the directorate of the company which monopolizes the local street railway service. It seems strange, in the light of the unsatisfactory experience of Philadelphia, that so near a neighbor as Baltimore should tolerate for a moment the proposition now under consideration there for similar municipal representation on the board of directors of the Consolidated Gas, Electric Light & Power Co. Judging from a very able discussion of the subject in the Baltimore "Sun" of June 28, it would seem that strong arguments are being put up for this plan, and as strong arguments as possible are being advanced against the regulation of this and other municipal franchise companies by a public-service commission like those which are giving so much satisfaction in New York and Wisconsin. We trust that wiser councils will prevail at Baltimore, and that the people and officials of that city will have none of such an unfortunate mixture of private and municipal control as invariably results when a few directors chosen by the city are placed in the board of a public-service corporation.

Should street car tracks on a double-track line be spaced far enough apart to permit a man to stand between the tracks with cars passing on either side, or should the tracks be close together, leaving only necessary clearance between the cars?

There is no uniformity of practice in this matter. In New York City, for example, there is ample space to stand between the tracks, while in Chicago there is with the widest cars in use only 8½ ins. clearance between them. Several fatal accidents to people caught between two passing cars have brought the width of the "death zone" as the newspapers have dubbed it, into general notice. There are, of course, good reasons for keeping down the width between the car tracks since that space is little used by traffic on any line where cars run frequently; but to reduce this space below the width necessary to permit a man to stand safely with cars passing on either side appears likely to cause not only deaths to pedestrians, but injuries to passengers who may carelessly allow arms or feet to project from the inner side of the car, particularly in summer, when open cars are run. The outcome of the current public discussion in Chicago will apparently be the spreading of track centers on the reconstruction work now in progress. Present ordinances permit tracks to be spaced 9 ft. 8½ ins. c. to c. By increasing this to 10 ft. 2 ins. and reducing the width of cars by 3 ins., a clear space of 20 ins. between tracks can be secured. This will not be sufficiently wide to encourage wearers of 1909 millinery to stand between passing cars, but is wide enough to permit one caught between the tracks with cars passing on both sides to escape without bodily injury.

Now that the most progressive towns and cities, and many rural sections as well, have undertaken to suppress the road-dust nuisance, which has become almost intolerable with the advent and wide use of the automobile, it is of great importance that road and street authorities get more light on methods of dust control and prevention than most of them seem to have. There is great danger that considerable sums of money will be wasted in costly experiments with various proprietary dust layers which have little to recommend them save an attractive name.

As a rule, the ordinary road authorities in the smaller municipalities of the country know absolutely nothing of the technical requirements for dust prevention. They are thus at the mercy of first one and then another smooth-tongued agent or distributor of well-written and handsomely illustrated advertising literature.

Much money would be saved and much nuisance avoided if these well-meaning but ignorant road authorities would entrust the matter of dust prevention to some competent engineer. In such an event, the first thing the road authorities would learn would be that most of the dust layers are mere palliatives, for which the best that can be said is that they do not have to be applied so often as water to lay the dust. If wisely advised, these road authorities would also learn that the fundamental principle in dust prevention is to construct a roadway which is as far from dust-producing as the modern science and art of preparing and using road materials can make it.

For existing macadamized surfaces which still have good wear in them, dust layers rather than dust preventers will of course be necessary for longer or shorter periods; but the aim should be to reconstruct these roads on the dustless principle as early as possible. Of course there may be some roads which will not warrant more expense than that involved for the best of the old-fashioned macadam construction, just as there are still other roads the travel on which does not warrant even that. Here again the services of an engineer of good training and judgment are required, in order that local road authorities may not go wrong.

Among the several means now being tried to re-establish democratic government in American cities are the initiative and referendum. The initiative permits some specified number of the legal voters of a city to secure a vote at either a special or regular election on any desired measure. The referendum either requires all or some ordinances passed by the city council to be ratified by the people, or else makes such a submission mandatory on petition of a certain number of voters.

The referendum of the first sort has long been in use throughout the country, although generally limited to such important matters as the approval of large bond issues or of franchise grants to public-service corporations. Neither the power of demanding a referendum vote nor the power of initiating legislation has often, if ever, been granted to the people of Eastern states—with the exception, if our memory serves us right, of the State of Maine. Both the referendum and the initiative are now quite common in the West and are becoming more so every year.

One of the arguments against the initiative and the referendum has been that either or both would break down of their own weight through the large number of questions that would be subjected to popular vote. The best test of the truth or falsity of this argument which has come to our attention recently occurred at Portland, Ore., where 35 questions were voted upon. Of these, 13 were carried and the remainder defeated. The aggregate vote on most of the questions was approximately 15,000, as compared with the total mayoralty vote of 18,000. The majorities in favor of some of the measures that were carried were as follows: requiring holders of franchises to file quarterly reports, 5,619; woman's auxiliary to police department, 975; placing burden of proof on all discharged civil-service employees, 2,279. Majorities against some of the 22 defeated measures were as follows: vehicle tax ordinance, 574; prohibiting use of patented article or process in street improvement, 9,208; payment for water mains by benefited property, 6,890; Des Moines plan of city charter, 5,740. It may be noted that of the questions mentioned above, the one which brought out the largest number of votes was that providing for the Des Moines or commission plan of city government, the total vote in this case being 15,612. The total number of votes cast in the matter of payment for water mains by benefited property was 3,848 for and 10,738 against.

We are indebted to "The Public," of Chicago, for the summary from which the foregoing figures were taken. Commenting on the votes on the 35 questions, "The Public" says that the result showed "purposeful discrimination throughout, even though valuable proposals were defeated." Although there is plenty of opportunity for difference of opinion as to what are and what are not "valuable proposals," we think no one could successfully question but what our contemporary is right in saying "purposeful discrimination" was shown in the Portland election. Had all the measures of an unusual or revolutionary character been carried, it might have appeared that the decisions were those of a mob rather than of persons who had well-defined opinions and were not afraid to express them.

Perhaps some of the enthusiasts for the Des Moines or commission plan of city government will think it strange that it should have been defeated at Portland. But to our mind it does not seem at all strange that a city of such size should hesitate to make such a radical change in its form of government, particularly when the voters already had municipal affairs so largely in their own control through this very right to exercise the initiative and referendum.

Of numerous recent schemes for securing more efficient municipal government in American cities, perhaps none is more remarkable than the creation of the position of General Manager by the City Council of Staunton, Va. The details of this plan and of its operations are given in an article by a citizen of Staunton, which we reprint elsewhere in this issue.

As the General Manager plan has been worked out under the local conditions which prevail at Staunton, that official relieves the City Council of a vast mass of executive work which it formerly did, or was supposed to do, besides unifying and harmonizing many of the activities of the city, saving duplications, effecting economies in the purchase and use of supplies and in the utilization of labor. In addition, the General Manager is credited with having introduced va-

rious much-needed reforms in accounting practices.

The General Manager is also the City Engineer of Staunton (7,285 population in 1900). This may work admirably in a small city, but in a larger place the duties of either office would demand all the time and strength of one man. As a matter of fact, the city engineers of some cities are to a large extent general managers of the whole city. The most notable example of this sort that has come to our attention exists at Seattle, Wash.

The Staunton experiment deserves watching. It affords a good illustration of what might be done in almost any city, without constitutional amendment or legislative action, if only there were an intelligent determination on the part of mayor and council to throw partisan politics and personal advantages to the winds and secure an efficient, readily accountable business administration. By such means, be they whatever local conditions or preferences may dictate, city councils become policy-determining bodies, and executive or administrative functions are entrusted to men chosen because of their fitness to execute them.

In this issue we present two papers describing the reinforced-concrete caissons invented by Maj. Judson and extensively used by him when stationed on the Great Lakes station and also a peculiar torpedo station built on the Mediterranean coast of France. Before this we have commented upon the engineering possibilities of this novel form of construction, the ease of design and erection and the admirable results attained in service. It is but another illustration of the desirability of adapting the cellular form to reinforced-concrete design.

Heretofore, however, we have not been able to procure any figures of cost such as are given for the Algoma breakwater by Maj. Judson. These figures add another advantage to the reinforced-concrete caisson made on shore and floated into place. It will be noted that the estimate made by the engineer corps, from large experience on similar work, was $105.18 per lin ft. for the stone-filled wooden cribs on pile foundation and capped with a standard concrete superstructure, a type of breakwater very common on the Great Lakes and subject to more or less maintenance and only medium life. The estimate for the caisson breakwater was $103.74 per lin. ft., but it was actually constructed for $78.29 per lin ft., a saving of 25.57% over the old type. Furthermore, Maj. Judson thinks that when contractors become acquainted with this style of work the unit cost will be materially reduced. So far there has been no opportunity to judge of the wearing qualities of the concrete caisson breakwater, but there is no reason why its limit of service should not be greater than that of the timber founded structure, and with this considerable saving in cost, its use should become quite general.

The sight of perspiring passenger conductors and brakemen in these hot July days, clad in the blue woolen uniform which the regulations require, leads one to inquire why American railway officers do not wake up to the fact that their trainmen would look better, would be vastly more comfortable, and would save money in the long run, if khaki uniforms were worn during the summer months.

With all the talk about the red tape and slowness of Governmental methods and the tendency to adhere to past customs and traditions, it is of interest to record that Uncle Sam has found the great advantage of dressing his soldiers and numerous other employees in uniforms made of cool and washable materials during hot weather. The railways, on the other hand, continue the use of the same old blue woolen cloth for summer and winter alike, and this applies to street railway companies as well as steam railways. There may be instances where some progressive manager has awakened to the advantage of dressing his force in different uniforms in July from those prescribed in January; but they do not happen to have come under our observation.

Concerning the Wear of Roads By Automobiles.

A dozen years or so ago, prophecies were common that a golden age for roads and streets would come when horses should be displaced by self-propelled vehicles. The horse, it was claimed, was the chief instrument in the wear of our roads and highways, by digging up the surface with the calks on his shoes. The iron tire of horse-drawn vehicles, too, was referred to as a crushing mill which was continually reducing to powder the material of the roadway surface. "Only give us vehicles without horses, with the wheel treads fitted with rubber tires," it was said, "and the wear on the roads will be reduced practically to nothing."

It is interesting to reflect on these ideas so commonly held a dozen years ago, and to compare them with the actual experience with the automobiles of the present day. It is doubtless well known to all our readers now that the wear upon highways due to automobiles is to-day the most serious problem facing the road constructor. Elsewhere in this issue we publish an abstract from a German contemporary, showing the result of a concentrated traffic of passenger automobiles upon a short section of highway in Germany. The article will be read with interest, we are sure, by every engineer interested in highway construction. It is about the most graphic example of the serious damage done by automobile tires to road surfaces that we have seen published anywhere. Yet, around the suburbs of any of the larger cities one can find almost equally serious examples of wear at any place where automobile traffic is concentrated, particularly where high speeds are possible and around curves.

The question may well be asked: why should the theory of a dozen years ago be so far apart from the experience of to-day? Why does the automobile tire, which was expected to produce no wear at all upon roads, actually wear them so terrifically?

It is worth while studying this question, because when the answer is found it is an excellent illustration of the fact that really theory and practice are not in conflict, as might at first sight appear. The only trouble with the theory of a dozen years ago was that it did not take all the facts into consideration. Hindsight is proverbially better than foresight; and, now that we know how badly the automobile wears our road surfaces, it is not difficult to see why this wear occurs.

In the first place, one important factor is the driving or propelling action of the automobile wheels. With horse-drawn vehicles, of course, there is no driving action by the wheels, and they exert only a downward pressure or lateral pressure on the road. The rear tires of an automobile, however, exert not only a downward pressure upon the road surface, but a powerful tangential push to the rear. The average power of the automobiles in common use to-day is probably from 15 to 40 HP., and the speeds, outside of city streets, will average not less than 15 to 30 miles per hour. If we consider the elements of a rubber wheel-rim in contact with the surface of the ground, Fig. 1, it may be easily seen that the point A, which has just come into contact with the road surface, exerts no tangential action upon the road. As the wheel moves forward from A to B, however, this tangential push of the surface of the rubber tire against the road rapidly increases from zero at A to a maximum at B, and as the tire lifts from the road at B and the pressure is released, the tendency is to push backward the material of the road just beneath it.

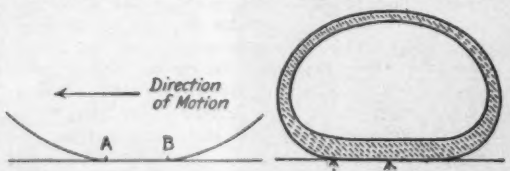

Fig. 1.

Fig. 2.

There is another reason why the automobile tire rapidly digs a rut for itself on any road where automobiles follow each other and keep nearly in the same track. The vehicle fitted with iron tires has a flat tread; the rubber tire, either pneumatic or solid, has a tread which is more or less rounded. Suppose, in a given automobile wheel, Fig. 2, the diameter at the center of the tread at C is 30 ins.; then the diameter toward the sides of the tread at D may be ¼-in. or ½-in. less, depending upon the pressure to which the tire is pumped, the contour of the tire cross-section, the weight carried by the automobile, etc. If we take only ¼-in. difference in diameter at the points C and D, we have a difference in circumference between the central part of the tread and the sides of the tread of about ¾-in. That means that at every revolution of the wheel, either the center of the tread must slip ¾-in. forward, or the sides of the tread must slip ¾-in. backward; that is to say, there must be, theoretically and actually, this difference in the relative movement of the center and the sides of the tread upon the roadway. Actually, of course, this slip is going on all the time in the movement of a rounded tire. There is some point between the center and the side where no slip occurs, and toward the center or toward the side there is all the time a slight slip forward or backward. As a result of this action, the round-tired vehicle has more rolling friction than a vehicle with flat tires. Of course, on an ordinary roadway such as a brick pavement or a smooth macadam, the pneumatic tire, notwithstanding its rounded surface, may move with little friction because the small stones and similar obstructions which would be crushed by the iron tire will simply indent the pneumatic tire. On an ordinary dirt road, however, the rounded rubber tire has more resistance to traction than a flat iron tire.

It is probable that the wear of the rubber tire on the roadway is more rapid as a rut begins to form and the width of contact of the tire with the road is thus increased. In the rut the sides of the tread—points where the tire diameter is an inch or even two inches less than at the center—may be in contact with the road. If the center of the tire moves in the rut without slip, the sides of the tire where the diameter is 2 ins. less must slip 6 ins. forward against the sides of the rut at every revolution. Thus the retarding action of the sides of the rut tends to increase the tangential force of the center of the tire upon the roadway and to dig the rut deeper.

We are aware that the tearing out of the macadam surface by the pneumatic tires of an automobile has been frequently ascribed to the "suction" of the tire upon the roadway. We do not see, however, in view of the foregoing explanation, that any "suction" hypothesis is needed to explain the disintegration of a macadam road surface by automobiles. Of course there is a wind current—an upward and forward suction—produced at the rear of the tires of a fast running automobile, and this wind current raises the fine dust loosened by the tire from the surface and sends it high in the air. Observation of this easily seen phenomenon has given rise to the common idea that the suction of the wheel on the road surface is what loosens the stone. Some such suction there doubtless is and it may possibly be a slight contributing cause of destruction; but any force exerted in this way must be very small compared with the terrific backward push of the rubber tire treads.

In the light of the above discussion it is evident that little can be hoped for in the way of help for the roads through changes in automobile construction. Flat steel tires in place of pneumatic tires might possibly reduce the wear upon the roads; but the success of the automobile is dependent on noiseless and easy-riding rubber tires. Further, we are only at the threshold of our troubles with highways due to automobile wear. For two years automobile factories have been working overtime, and it is said that 200,000 automobiles are to be built next year. The burden on taxpayers to build and maintain roads to carry automobile traffic promises to become so great that a road repair tax may have to be laid on the vehicles themselves.

LETTERS TO THE EDITOR.

Canadian Universities Having Courses Leading to Diploma in Public Health.

Sir: My attention has been drawn to an editorial in the issue of your paper dated April 15, 1909—"State and Local Health Administration in Ontario, North Carolina, and Great Britain."

In thanking you for your reference to this important matter, there is just one error I would like to correct. The remark to which I refer is:

The first is that there is not, in the United States at least—and we are pretty sure there is not in Canada, either—any adequate means of training health officers.

I am pleased to state that three of our leading universities have made provision for a special course by which graduates in medicine may secure a diploma in public health. These universities are University of Toronto, Queen's University, Kingston, and McGill University, Montreal.

You will thus note that the universities have anticipated any demand for this particular line of post-graduate work which would be sure to follow legislative enactment requiring that health officers of cities, districts, or counties would require such special qualification before appointment.

Yours truly,
Chas. A. Hodgetts, M. D.,
Chief Health Officer of Ontario.

[Since writing the foregoing letter Dr. Hodgetts has very kindly secured and forwarded to us the annual announcements of the three universities which he names. These announcements contain the requirements for the diploma of public health (D. P. H.) in each of the institutions. Dr. Hodgetts states that this year two candidates secured the D. P. H. at McGill. So far as Dr. Hodgetts knows, the degree has not yet been conferred at either of the two other universities. Dr. Hodgetts says:

Unfortunately there is no inducement for medical men to take up this important branch of medicine—indeed to my mind the most important one in preventive medicine.

We quote from the McGill University Catalog, as follows, including information regarding other special instruction in sanitation and related subjects as well as that designed for holders of the degree of M. D. who wish to obtain a D. P. H.:

In the session 1899-1900 the Faculty instituted a post-graduate course in Public Health and Sanitary Service and since that time other courses as described below have been instituted.

Special instruction is given in this department, leading to the Diploma of Public Health; also for Engineers, Architects, and those wishing to include this subject in their final examination for the degree of Doctor of Philosophy (Ph.D.).

(1) DIPLOMA COURSE IN PUBLIC HEALTH.

Candidates undertaking this course must have possessed a Degree in Medicine, or other qualification for practice, for at least twelve months before he is competent to receive the diploma. The course prescribed is as follows:

(1) A course of lectures in Public Health (to be omitted in the case of candidates who have attended such a course before graduation).

(2) A three months' course in Bacteriology, special attention being directed to the pathogenic organisms and parasites—such course to be omitted on presentation of proof that it has previously been taken.

(3) A six months' course of practical study of outdoor sanitary work under a medical officer of health (to be omitted in the case of medical health officers holding appointments prior to the establishment of this diploma course).

(4) Three months' attendance and clinical instruction at a hospital for infectious diseases (unless such course has already been taken prior to graduation).

(5) Three months' instruction in sanitary Chemistry and Physics, with practical work in a chemical laboratory.

The examination for the Diploma shall cover the following subjects: Examination of clinical cases at an infectious hospital; the drawing up of outlines for annual and other reports of officers of health; a report upon the sanitary condition of some actual locality; the chemical analysis of liquids and gases and of specimens of food; demonstration of the consideration and use of meteorological, hygienic and sanitary apparatus; microscopical examination of specimens submitted; description of specimens of human and other diseased tissues; practical examination in the employment of the usual bacteriological methods; the inspection of carcasses of animals to be used for food.

The above examination shall be written, oral and practical, and shall extend over a period of four days.

The following is a list of subjects included in the curriculum of study:

(a) *Sanitary Chemistry*: Examination of air, gases, water, the action of water on metals, milk, food and beverages; detection of poisons in articles of dress and of decoration; the chemistry of sewage.

(b) *Sanitary Physics*: Principles of statics, pneumatics, hydraulics, light and photometry, heat and thermometry, the principles of hygrometry, (only in their application to hygiene).

(c) *Sanitary Legislation*: Statutes and by-laws relating to public health; the powers of public sanitary authorities.

(d) *Bacteriology and Parasitology*: Modes of propagation of disease and transmission of disease between man and man, and man and animals; bacteriological analysis in relation to public health matters; natural history of microbes and animal parasites.

(e) *Vital Statistics*: Calculation and tabulation of returns of births, marriages, deaths, and diseases.

(f) *Meteorology and Climatology*, including the geographical and topographical distribution of disease.

(g) *Preventive Medicine and Practical Sanitation.*

The fee for the Diploma, including laboratory fee, shall be $50.

(2) COURSE FOR CIVIL ENGINEERS.

This course is given to meet the requirements of engineers, particularly those making a specialty of sanitary engineering.

The object of the instruction is to elucidate the public health principles involved in engineering problems, *e. g.*, ventilation, water supplies, sewage disposal, and drainage systems.

(3) COURSE FOR ARCHITECTS.

Special instruction is given in those branches of public health relating to Architectural Work, *e. g.*, lighting and heating, ventilation, sanitary fixtures, draining and plumbing.

(4) COURSE FOR THE DEGREE OF DOCTOR OF PHILOSOPHY (Ph.D.).

Hygiene, or some particular branch of it, may be taken out as a minor subject in the final examination for the Ph.D. degree. Special arrangements are made to suit the student in order that the work done in this department shall be a supplement to his major subject taken out in Applied Science.

(5) COURSE FOR PROMOTION IN THE ARMY MEDICAL CORPS.

As hygiene forms one of the compulsory subjects in the examination for promotion in the Permanent Army Medical Corps, special classes are held for the purpose of giving instruction in this subject—particular attention being paid to Military Hygiene.

The attendance in this class counts towards the requirements for the Diploma of Public Health.

Courses (2) and (3) can be commenced at any time during the session, and usually are of about three months' duration.

A small fee will be charged for each of the Courses (2), (3), (4) and (5).

In general, the D. P. H. requirements at McGill are similar to those in Great Britain.

It is interesting to note that a Chair of Hygiene and Public Health was founded in the medical school of McGill in 1871 and endowed in 1893, thus antedating the D. P. H. course by many years. The present occupant of the chair is Prof. T. A. Starkey.

Candidates for a D. P. H. at Toronto University must be graduates in medicine and must attend two summer sessions of three months each and one winter session of six months, but the work of the latter may be combined with fifth-year work for M. D. at Toronto (some of it is required, anyway). The first summer session is devoted to laboratory work in sanitary chemistry, in bacteriology and in parasitology. The winter session is occupied with hygiene, sanitary engineering, and other topics similar to the class and lecture room work at McGill. At the conclusion of the winter session examinations are held on the work done during the previous two sessions. Only those who pass these examinations are permitted to enter the second summer session, which is

devoted to a course of Practical Work in Public Health under the supervision of the Provincial Board of Health, including the methods of dealing with infectious diseases, inspection of schools and other buildings, factories and dairies, inspection of water-supplies and sewage disposal plants and other forms of municipal sanitation.

At Queen's University, Kingston, Ont., two classes of D. P. H. candidates are recognized: those holding B. Sc. and M. D. degrees, and those holding the M. D. degree only. We quote as follows:

COURSE FOR DIPLOMA OF PUBLIC HEALTH (D.P.H.)

A. For candidates who have taken the B.Sc., M.D. course.

Such candidates will be entitled to enter for examination for this Diploma on presenting certificates of having taken: (1) Three months in attendance and clinical instruction in an Hospital for Infectious diseases; (2) three months in a Bacteriological Laboratory, devoted to bacteriological aspects of Public Health; (3) one week in practical testing of milk and milk products for chemical constitution and common adulterations; (4) six months with a recognized Medical Officer of Health in the practical study of Sanitation.

B. For candidates proceeding to take this Diploma after graduation as M.D.

Such candidates will be entitled to enter for examination for this Diploma on presenting certificates of having taken:

(1) Three months' course in Sanitary Physics (Principles of Statics, Pneumatics, Hydraulics, Light, Photometry, Heat, Thermometry, Hygrometry).

(2) Three months' course in Sanitary Chemistry, especially devoted to Quantitative and Qualitative analysis of air, water, and common food stuffs. This course to include one week's work of practical testing of milk and milk products for adulteration or sophistication.

(3) Three months' course in a Bacteriological Laboratory devoted to bacteriological aspects of Public Health Work, such as, examination of sputum, blood, swabs, water and milk, and the detection of common animal parasites.

(4) Three months' course in advanced Hygiene, covering especially a discussion of sewage and garbage disposal, water supplies, disinfection, transmissible diseases, vital statistics and sanitary legislation.

(5) Three months' course in Sanitary Engineering, including water services, sewerage systems, sewage and garbage disposal.

(6) Three months' attendance and clinical instruction in an hospital for infectious diseases.

(7) Six months with a recognized Medical Officer of Health in the study of practical sanitation.

Apparently, about a year of special work is required for the first class of candidates, but it is not clear how much time is demanded for the second class, possible simultaneous courses being considered.

It is greatly to the credit of these three Canadian universities that they offer so good post-graduate courses in public-health work to their doctors of medicine, but little credit is reflected on the municipal authorities and citizens of Canada by the scanty use made of these facilities at McGill and their apparent non-use at Toronto and Queen's. Unquestionably, instruction so generously offered would be seized upon if there were any material demand for men of such training in Canada. Conditions in the United States are even worse, so far as medical schools are concerned. When will these two great American countries awake to the need of health-administrative reforms?—Ed.]

Tests on the Steel Transmission Towers Which Failed on the Line of the Great Western Power Co.

Sir: Referring to an article in Engineering News, May 13, 1909, p. 515, entitled "The Failure of a Portion of a Steel Tower Electric Transmission Line in California," by Edwin Duryea, Jr., M. Am. Soc. C. E., I would like to call attention to an error in a caption to a cut, and also add a few remarks concerning the subject in general.

Fig. 3 in that article is described as follows: "Collapse of One of the Towers During Construction, Due Probably to Improper Temporary Guying." The photograph looks much like one of those taken at the time of a test made under the direction of Galloway & Markwart, Civil Engineers, who were engaged to report upon the probable cause of failure of the towers. Two tests to destruction were made on towers erected in the hills back of Oakland near the Great Western Power Co. substation. The four legs of the tower were surrounded with concrete so as to insure a failure at some point above the anchorage. The load was applied to all three cross arms in a horizontal direction at right angles to the direction of the transmission line, the test tower being so situated that it was possible to pull from a side hill across a canyon at about the same elevation as the top of the tower. The pulling was done by means of chain blocks and measured with a calibrated dynamometer.

The first tower tested was a standard tower without alterations. A total pull of 8,400 lbs. on the three cross-arms buckled the main angle legs of the tower just below the lower cross-arm as shown in Fig. 3 of the article above mentioned. On close observation one can see the three cables attached to the cross-arms, also on the opposite side of the tower, a guy wire to keep same from falling completely over.

The second tower tested had previously been strengthened by inserting horizontal members in the "basket" and diagonals and horizontals in the first two panels below the basket; also there was some further strengthening near the bottom, but this latter was of doubtful value. A total pull of 11,250 lbs. applied in the same manner as Test 1 buckled the legs of the "basket" on the compression side just above the lower cross-arm.

An article in Engineering News, March 12, 1908, p. 282, entitled "Steel Towers for High Tension Transmission," states that these towers were designed for simultaneous side pulls of 1,000 lbs. at the top and 2,500 lbs. at each of the three cross-arms, with a vertical load of 1,000 lbs. on the end of each of the six cross-arms, and at the end of any one cross-arm a pull of 3,000 lbs. in the direction of the line. These assumptions were made as a result of full-size tests. The article also states that ordinary tamped earth foundations would be sufficient, but this has been proven otherwise as shown by the failure.

The reasons for referring to this latter article is to compare the assumptions that were made in the designing of the towers and the results of these recent tests made by the Great Western Power Co. The total load of 8,400 lbs. in Test 1 is nearly the same as a load of 1,000 lbs. at the top of the tower and 2,500 lbs. at each of the three cross-arms. The towers seemed to have been designed with a small factor of safety.

An anlysis of the requirements of the tower might be interesting. As stated by Mr. Duryea, the towers carry two circuits of three stranded cables each and one stranded steel "ground wire." The former are about ½-in. diameter and the latter about ⅝-in. diameter. In common practice the force of wind on a smooth cable is assumed to act on one-half the diameter. It therefore seems reasonable to assume for stranded cables that three-quarters of the diameter will be exposed to the wind's force. Using 30 lbs. per sq. ft. and considering the length 750 ft. the copper cable will exert a force at each tower tending to overturn, of 703 lbs. and the steel cable 527 lbs. There is in addition about 140 sq. ft. of the tower itself exposed to the wind. This includes both sides since the tower is so open that the windward side affords no protection to the leeward. This produces a total wind pressure of 4,200 lbs. distributed practically uniformly from the ground to the top of the tower. To arrive at a probable safety factor of the tower, these external forces that the tower would be subjected to must be compared with the test forces applied. Since the effect of a force applied at the top of the tower is twice that produced with the same force applied uniformly from the bottom to the top, 4,200 lbs. distributed is equivalent to 2,100 lbs. applied at the top.

A summary of the forces the tower would probably be subjected to in a 30-lb. wind is as follows:

	Lbs.
One steel cable	527
Six copper cables at 703 lbs.	4,218
Force on tower	2,100
Total	6,845

This force is not exactly comparable with the test force applied because in the test there was lacking the weight of the transmission cables, which produces a moment to counteract the moment of the wind. An equivalent moment is produced by a force of 345 lbs. applied at the top of the tower. The total wind force would then be 6,500 lbs. and is comparable now with the force of 8,400 lbs. applied in Test 1.

$$\text{Therefore, safety factor} = \frac{8,400}{6,500} = 1.3 \text{ in Test 1.}$$

After reinforcing tower as previously described the

$$\text{safety factor} = \frac{11,250}{6,500} = 1.73.$$

The above figures are based upon a 30-lb. wind which to produce requires an actual wind velocity of 77.5 miles per hour, using the formula $P = 0.005V^2$. Assuming less wind per sq. ft. would of course increase the safety factor, but a hurricane of 77.5 to 80-mile wind is possible and 40, 50 and 60-mile winds are probable. Safety factors at other wind velocities for the tower as designed, assuming a concrete anchorage would then be as follows:

Miles per hr.	Lbs. per sq. ft.	Safety Factor.
77.5	30.0	1.30
60.0	18.0	2.17
50.0	12.5	3.12
40.0	8.0	4.88

The question then arises as to what are reasonable assumptions to make. The writer believes the safety factor should be not less than three on the basis of 30-lb. wind which may occur. A safety factor of three is little enough considering the importance of keeping power on the line. A failure means a loss in prestige and standing with the power consumers, and incidentally a money loss. A steel tower line is supposed to be something more stable than an ordinary wooden pole line and it should be safe under all wind conditions.

In bridges it is common to allow four or five, in buildings four, so in towers of this description three is necessary. This might be reduced if proper account could be taken of the dynamic action of the wind and the shock due to the swaying of the wires and from gusts of wind on the towers themselves.

The question of safety factors and assumptions depends largely upon the designer. Many will gamble upon the chance of the worst conditions not arising, but one failure is enough.

Referring to the tower as designed and the failure at a test load of 8,400 lbs., this tower has a safety factor of 2.5 on the basis of a 57.5-mile or 15.5 lbs. per sq. ft. wind, assuming a concrete anchorage of sufficient strength which has since been provided under all earth anchored towers.

The immediate cause of failure as stated by Mr. Duryea was a weakness in the foundations instead of in the superstructure and he omitted discussing the latter, but since the matter has been brought up a few remarks are in order. There is little or no bracing provided to resist horizontal shear, the tower being really four independent braced legs connected at the top and center and is hardly susceptible of exact analysis. It would seem that the ordinary standard design of four legs and a series of panels with diagonals and horizontal members would be a more logical scheme.

Yours very truly,

A. H. Markwart, Assoc. M. Am. Soc. C. E.
Galloway & Markwart, Civil Engineers.
Balboa Building, San Francisco, Cal., May 27, 1909.

Annual Convention of the American Institute of Electrical Engineers.

The twenty-sixth annual convention of the American Institute of Electrical Engineers was held at the Hotel Frontenac, in the Thousand Islands of the St. Lawrence River, June 28 to July 1.

The relaxations of the convention were numerous and favored by fine weather. While the number present was possibly smaller than would have been attracted to a more central location, yet those present probably profited more than could have been possible further south under the prevailing weather conditions. The total registration of members and guests on the last day of the convention was 265.

A golf tournament was held for the men and a bridge-whist tournament was arranged for the ladies. For those not inclined to these diversions, there were excursion parties about the islands and ample chance was offered for fishing and boating.

The entertainment program did not interfere with a long program of 36 papers. As nearly all of these had been printed and in the members' hands for several weeks, they were presented in brief abstract and most of the time of the sessions was given over to the discussions.

The first session took up experimental work largely—the convection of heat from wires, testing of transformers and transformer steel, a 100,000-cycle, 2-KW. alternator. The second session was devoted to alternating-current motors and the papers were mostly on the development of so-called "induction" or "repulsion" motors, of ½ to 10 HP., of variable-speed characteristics similar to those of the direct-current shunt motor. These papers and the discussions provoked showed the attempts of the designing engineers to get around the inflexibility of the common induction motors for industrial power.

The third and fourth sessions were devoted to power transmission and to high tension developments—long-distance lines, armored cables, high-tension surges, etc. The fifth and sixth sessions were in parallel, an arrangement which worked some hardship on those interested in the allied subjects presented. In one of these parallel sessions, the use of various electric motors in steel mills was discussed and in the other, the subjects included miscellaneous points of design, such as heavy fuses, 25 and 60-cycle alternators, auxiliary-field poles, iron losses, etc. At the seventh and last session two papers on design were scheduled with two on education. One took up the training of shop men and the other was on the places of the classics in engineering education.

The presidential address, made at the opening of the convention, by Mr. L. A. Ferguson, of Chicago, was on "The Centralization of Power Supply." In this address the author dwelt upon the tendency to centralize the production of electrical energy, wherever possible, in a few large plants rather than to distribute the generation with numerous small units. The first point in the argument was the economy of producing electric current in large quantities, as with most manufactured products. After this came the increased ability to market the securities of the larger companies having a more diversified arrangement of customers and greater concentration of administration. By centralization of production and diversity of loading, by combining widely different classes of service, the load factor had been bettered—the ratio of maximum load to average had been made nearer unity and the fixed charges per unit output had been reduced. Further improvement was believed by the author to be possible.

By this centralizing process the necessary emergency-reserve units could be made in large sizes with a resultant decrease of reserve-capacity costs per KW. Centralization of power plants was held to be a forward step in civic improvement through the abatement of smoke. The greater opportunities of watching the combustion processes in large plants to prevent beclouding the atmosphere, and the chances for more economical combustion were noted. Mr. Ferguson, toward the end of his address, spoke of the necessarily monopolistic nature of an electric-power supply business. He believed that this was becoming generally recognized so that the centralizing processes could go on and the large combined concerns could secure privileges under reasonable and proper public control.

Mr. L. B. Stillwell, of New York City, the president-elect of the Institute, was called to the platform and made a few extended remarks on the coming year's work. Three problems seemed to him to confront the Institute: (1) the provision for special sections to carry on work not of general interest, (2) the standing of engineers in the community and a wider recognition of their fitness to deal with certain public questions and of the value of their opinions in cases of public policy, (3) the need of a broader education for the young men now entering the engineering professions.

The contents of the numerous papers of the convention have been considered briefly in the following article.

Convention Papers: American Institute of Electrical Engineers.

The papers of greatest general importance to the profession, among the many presented at the annual convention of the American Institute of Electrical Engineers, have been briefly abstracted in what follows. These abstracts are grouped under general subjects and are not necessarily given in the order of presentation at the convention. The more important points in the discussions have been appended to their respective papers where it has been judged useful.

EXPERIMENTAL WORK.—"The Convection of Heat from Small Copper Wires," by Prof. A. E. Kennelly and Messrs. C. A. Wright and J. S. Van Bylevelt, of Harvard University, gave the details of an experimental investigation of the laws governing one form of heat dissipation. The work was divided into two classes: (1) on free convection where the wire is supported in still air free from movements except those produced by this heated wire alone, (2) on forced convection where there is relative motion of air and wire not caused by the heated wire itself.

In the first group of experiments, the wire was held horizontally in a sealed tank under different pressures. The wire was kept at constant uniform temperature by electric current and the amounts of heat dissipated were determined by measurements of the electrical energy furnished. In the second group of studies, the wire was mounted on a long rotating fork and the heat energy dissipated was determined in a manner similar to that of the tank experiments. The necessary electrical connections were made through collector rings.

The results of these tests showed that, with normal air pressure, the linear convection in

still air was proportional to the temperature elevation of the wire and was about 60,000 watts per linear centimeter per degree C. To a first approximation this is independent of the size of wire, though increasing slowly with the diameter. Under varying air pressure, the linear convection increased with the pressure by an exponential law.

For the condition of wind blowing against the wire, the convection was proportional to the temperature elevation. The linear convection loss for wind velocities between about 6 and 65 ft. per sec. increased as the square root of the velocity. At any single velocity the convection increased with the size of the wire, as a first approximation, directly proportional to the wire diameter. In arriving at these conclusions the corrections for heat radiated were applied.

In the paper, "Testing of Transformer Steel," Messrs. M. G. Lloyd and J. V. S. Fisher, of the U. S. Bureau of Standards, Washington, D. C., described a method of determining the losses in sheet-steel for cores of alternating-current transformers due to the reversals of magnetic flux therein.

The specimen to be treated was cut up into strips 10 x 2 ins. and assembled in four bundles, wrapped with friction tape and each slipped into a solenoid about 9 ins. long. These bundles are then arranged in a closed square with the ends of the iron cores joined by small iron corner pieces to give a good magnetic joint. Each solenoid is made up of two windings, the inner one of which is 90 turns of No. 20, while the outer is 250 turns of No. 14. The outer heavy coils are connected in series as a magnetizing coil. Two of the inner secondary coils are connected together and used to measure the actual magnetic flux in the iron core. The other two secondary coils are connected in series to the voltage coil of wattmeters through whose current coil the current for magnetizing the iron sample is led. This arrangement allows an easy method of finding the hysteresis and eddy current losses in this iron. The sources of inaccuracy are known and their magnitude is easily found. The method has been found more practical by the physicists of the Bureau than any hereto developed, and the accuracy of results is within 1% with a sample of 4.4 lbs. of iron.

Mr. L. W Chubb, of the Westinghouse Electric and Manufacturing Co., Pittsburg, in the paper, "Testing Transformer-Core Losses for Sine-Wave Results from Commercial Circuits," described an instrument which is

an indicator enabling the voltage to be so adjusted that the loss in a transformer coil as increased by an ordinary wattmeter will be the same as at normal voltage on a circuit having a sine-wave voltage of normal frequency.

It is well known, of course, that if the shape of the voltage curves of circuits on which a transformer is tested are not alike then the core losses vary, even though the effective value of the alternating voltage is the same in each case. This instrument is called an "iron-loss voltmeter," and is essentially but a combination of a small choke coil, with an iron core, and a wattmeter. The wattmeter indicates the power supplied to the choke coil and this varies for different voltages, frequencies and voltage-curve forms. By calibrating the meter dial for different voltages and a normal frequency on a circuit whose curve of alternating voltage is a true sine wave, the instrument will show, on any commercial circuit of normal frequency, what sine-curve voltage would give the same losses in the core of a transformer as does that actual voltage under consideration. The inaccuracies due to frequencies of current alternation other than normal, for transformer cores having a different quality of iron than the choke-coil core, and for transformers having a different ratio of hysteresis and eddy-current losses than the new instrument were shown to be practically smaller than inaccuracies in other forms of commercial testing.

In the discussions on this paper, many expressed doubts of the possibility of using the instrument widely in common practice with the accuracy claimed. In rebuttal the author and others cited the experience of the Westinghouse company which has adopted the instrument.

ALTERNATING-CURRENT MOTORS.—The discussion of variable speed motors was opened by the paper, "A Theory of the Adjustable-Speed Single-Phase Shunt Induction Motor," by Mr. F. Creedy, of Pittsburg. This type of motor considered was essentially an annular stator, having an imbedded winding used with a direct-current motor armature, carrying the usual commutator. For a two-pole machine, four sets of brushes rested on the commutator, when one pair is considered parallel to the resultant stator-field poles, the other pair is perpendicular. Each pair is commonly short circuited. Instead of short circuiting both pairs of brushes, the author connects one pair to the secondary winding of a transformer whose primary winding is on the main supply. This impresses added voltage in the rotor which would unbalance the voltage conditions did not the rotor speed up enough to preserve the balance. The author's second method of speed control was to effect a similar unbalance by inserting inductance or capacity between the other two brushes. To preserve the voltage balance the rotor again changes speed. A third method was suggested by putting an additional winding on the stator or field to which one set of brushes would be connected, changing the field magnetization. The theory of these schemes was shown by vector diagrams and confirmed by the study of experimental results. The author concluded that small motors of ½ to 10 HP. were possible with a speed variation of as great as 1 to 3, but that the machines would be expensive and of low efficiency (70%) compared with direct-current variable-speed motors.

Mr. E. F. W. Alexanderson, of Schenectady, in his paper, "A Repulsion Motor with Variable-Speed Shunt Characteristics," showed the results of independent study in which methods of control similar to those of Mr. Creedy, noted in the preceding paper, were discussed from a slightly different point of view. The important practical point in this second paper was that certain "repulsion" motors, already on the market, having a compensating field winding to improve their power-factor could be run as variable-speed motors with the simple insertion of a regulating transformer between the compensating and the main circuits.

The paper, "Multi-Speed Production Motors," by Messrs. H. G. Reist and H. Maxwell, of Schenectady, was a review of the methods of efficiently obtaining more than one operating speed of motors having "squirrel-cage" rotors. These methods were of three kinds: (1) changing the number of poles in the electrically revolving field by regrouping the windings, (2) changing poles by using different field windings in the same slots, (3) use of two motors on a single shaft, with the rotor circuit of the first not closed on itself but through the stator winding of the second motor.

In the paper, "Reduction in Capacity of Polyphase Motors Due to Unbalancing in Voltage," Messrs. S. B. Chanters, Jr., and W. A. Hillebrand, of Leland Stanford University, Cal., reported on an experimental study of the effect of unequal voltage change at the several terminals of polyphase motors and of change in the normal phase relations of the voltages at the terminals. It was found that, in general, serious overheating of the motors was caused, though the trouble was worse for phase shift than for voltage unbalance. With induction motors the heating with both phase shift and voltage unbalance was worse than would be expected in considering either alone. The reverse was true with synchronous motors. In the discussions of this paper many observations were reported of the equalizing action of synchronous motors in restoring the voltage balance, apparently by taking power from one phase and returning it to another.

The paper, "Heating of Induction Motors," by Mr. A. M. Gray, of Milwaukee, outlined specific computations to show a method of predetermining the heating in a design of induction motors. In "The Current Losses of the Single-Phase Induction Motor," Prof. A. S. Langsdorf, of St. Louis, presented a new mathematical demonstration of the action of this machine and derived a circle equation for the current relations.

POWER TRANSMISSION.—The paper, "Output and Regulation in Long-Distance Lines," by Mr. P. H. Thomas, of New York City, was a review of the design of transmission systems, of greater length and size than now known, in respect to the elemental qualities of line output, energy losses and voltage variation. All of their qualities are adversely affected by the self-induction, or electrical elasticity, and the static capacity, or condenser effect, of the transmission line. The author's efforts are directed then to the elimination of the effect of both. The static capacity of the line was considered to absorb and to repay a wave of energy, for each alternation of voltage, the maximum point preceding that of the voltage. So the author terms the energy of the capacity a "leading energy." Reversely, the energy given to, and received back from, the line inductance is called "lagging energy." When energy is being absorbed by the one it is being discarded by the other, so that if the energies can be made equal the effects of both inductance and static capacity balance and are neutralized. The proper balance of a line may be calculated and designed, for a particular loading only—since the leading energy depends on the voltage, and the lagging energy depends on the load of current. For balance large conductors are split into multiple and parallel arrangements of smaller ones. For light loads the balance may be partially restored by running synchronous motors on the line and feeding in the lagging energy needed. Where it is not desired to balance the inductance and capacity by splitting large conductors into small ones, it may be artificially done by feeding in the leading energy at either end or at intermediate points by synchronous motor sets. These motors can be made to draw either "lagging" or "leading energy."

For computing the constants of a transmission by more accurate methods than the present approximations in general use, Mr. Thomas developed equations in circular-trigonometry terms. These were presented in a companion paper, "Calculation of the High Tension Lines." Comparison of the exact with the approximate methods showed that for present practice the latter were sufficient, but that for extra long distances, higher voltages and greater power outputs, the approximations were not close enough. The discussion of these two papers centered largely in the methods of computation exposed in the second paper. Many claimed that it would be simpler to have used hyperbolic trigonometry and others maintained that such hyperbolic functions are not adapted for computation formulas.

In the paper, "Even Harmonies in Alternating-Current Circuits," Mr. John B. Taylor, of Schenectady, very briefly mentioned the lack of general knowledge concerning the influence of those variations of alternating voltage and current whose period was an even multiple of the main period of the alternating voltage or current. The presence of even harmonies may produce an entirely unsymmetrical wave of voltage or current, which may therefore prevail more in one direction than another—or may be said to have a "direct-current component." The same effect is produced in grounded transformer circuits by the inleakage of direct currents, as from railway lines. It was found that such currents caused a pronounced even harmonic in the wave of exciting current, but did not "bias" the transformers enough, magnetically, to cause serious heating in the windings. Electrolytic counter-e. m. f. cells had been proposed, but their use had not been found necessary.

Prof. J. B. Whitehead, of Baltimore, Md., in his paper, "The Resistance and Reactance of Armored Cables," presented the results of a few studies, on armored power cables, undertaken to settle the possibility of using single-conductor iron-armored cables for alternating currents in spite of an adverse general opinion. It was found, in brief, that the theoretical reactance of such a cable was from two to four times the unarmored, and the test results were about 7/10 the

theoretical. The actual impedence of double-conductor, iron-wire armored cable was found to be 5% (at 60 cycles) and 3% (at 25 cycles) greater than calculated values. The author concluded that the use of iron-armored single-conductor cables was practical.

"Losses, Induced Volts and Amperes in Armor and Lead Cover of Cables," by Mr. H. W. Fisher, Chief Engineer, Standard Underground Cable Co., Pittsburg, Pa., showed a parallel investigation on single cable with a double steel-tape armor and lead sheath. With the armor and lead sheath short circuited the impedence was 3½ times the common resistance, and with the armor and sheath as an open circuit the impedence was 13 times the resistance. In the discussion of these papers the views were generally expressed, and most strongly by Dr. C. P. Steinmetz, of Schenectady, that single-conductor cables for alternating-current transmission were not safe. Arcing between sheath and armor could be expected with a consequent destruction of the cable.

"Corona Phenomena" was the title of a paper by Messrs. W. S. Moody and G. Faccioli, of Schenectady, N. Y., in which they compared theoretical conditions, for the appearance of glowing brush discharges of electricity on high-tension alternating-current apparatus, with experimental. The studies reported in the paper were made with cylindrical rods of known diameters parallel to a metallic plate or inside metal rings. The voltages employed in the experiments varied from 24,000 to 100,000, the wave of voltage being very close to a true sine. Throughout the work great care was taken to work with clean and uniform surfaces of conductors.

In the simple case of rod and plate in air the shape of curves between voltage and diameter of rods agreed with that of theoretical curves, but the former curves were translated to a certain fixed value higher than the theoretical. Spots of insulating material on the wire began to glow before the voltage point of a true corona effect is reached and did not die out on reducing the voltage to the value at which the glow was started. To extinguish them the initial voltage was reduced some 12%. This effect of insulation was so peculiar that further investigation was made. When a tube or ring of any insulating material was slipped on the rod the brush discharges started at the edges of the insulation. At one of this points was the final breakdown (jump spark) and at a voltage some 20% less than for the normal breakdown without the insulating tube. By putting metallic disks at the ends of the insulation normal conditions were restored. In explanation of this phenomenon the authors cited the probability of a sharp difference in the specific static capacity of the field occupied by the insulating medium and by air, causing a transverse stress analogous to mechanical shear.

By placing short tinfoil cylinders on the rods the voltage at which the corona effects appeared was raised, the magnitude of change depending on the number and position of cylinders. The shorter the rod under study, the higher was the voltage found necessary to produce corona.

The paper related many allied experiments showing the great number of minor conditions that markedly affected the voltage of corona appearance. In conclusion the authors expressed their belief that the voltage at which corona appeared in apparatus of limited dimensions, like transformers, not only depended on the dimensions of the apparatus and the mutual action of the various parts, but also was a complex function of many quantities and conditions, not yet well understood.

"Surges on Cable Systems with Aluminum Cell Protection" was presented by Prof. E. E. F. Creighton, of Schenectady, and Mr. S. D. Sprong, of New York City, and described an investigation into the possibility of protecting the underground alternating-current cable system of the United Electric Light & Power Co., of New York City, from voltage surges by the use of aluminum-cell electrolytic arresters. Current is generated for this system at 7,500 volts, is transmitted to two substations and there stepped down to 2,250 volts, two-phase, for distribution on a three-wire system. The lighting load and half the industrial-power load are carried on the two outer wires at 3,000 volts. The voltage is regulated on the outer lines only and by a motor-operated induction regulator. This arrangement puts a large concentrated inductance in series with the electrostatic capacity of the underground cable system and gives conditions suitable for the development of abnormal rises of voltage, puncturing cables and breaking down the regulators. Abnormal potentials were by-passed around the regulator coils to the station bus bars by aluminum electrolytic cells and from there passed to the ground by other cells. In the studies reported, surges were caused by an arc between one line and the ground and while instruments showed the action of the protectors no rise of potential like that prevailing before was found.

The discussion on this paper turned into a catechism of how the aluminum cells were made and arranged for the several services to which they were being adapted. The aluminum cell was generally considered as the only available "electrical safety valve" for abnormal rises in potential.

"Modern Telephone Cable" was the title of a paper by Mr. F. B. Jewett, of New York City, in which he discussed a few clauses in the cable specifications of the associated Bell companies. The surprising statement was made that of the 8,000,000 miles of wire in the plants of the associated Bell companies, over 6,800,000 miles were in cable. A second statement, surprising to those not familiar with telephone engineering, was that this development had come about in spite of the fact that one mile of cabled circuit of the common No. 19, B. & S. gage, attenuated telephone currents, and speech, as much as 35 miles of No. 8, B. & S. gage, copper, open-wire, toll line. The use of cable has been necessary, however to concentrate the great number of telephone circuits coming into exchanges in all but the smallest places.

The requirements of a modern telephone cable as outlined by Mr. Jewett provided (1) maximum number of circuits in a minimum space, (2) maximum transmitting efficiency compatible with other requirements, (3) sufficient insulation strength to carry normal operating voltages and to protect against lightning and high-tension contacts, (4) easy maintenance, (5) moderate cost. These requirements are best met with in cables in which copper wires are covered with paper ribbon, twisted in pairs, bunched, wound with paper ribbon and covered with a lead sheath. The function of the paper is that of a mechanical separator and dry air is the real insulator. It is advisable to use air rather than rubber or some similar insulator on account of the much smaller static capacity and hysteresis losses of the paper cables for the telephone currents whose frequencies range into the thousands of cycles per second. A minimum static capacity and hysteresis loss are far more important in keeping down the attenuation of speech than is a high insulation strength.

ELECTRICITY IN STEEL MILLS.—The electric drives in steel mills, supplanting steam drive and hydraulic control are considered among the greatest achievements of the electrical and industrial engineers and to give the electrical engineers generally a better insight into the character of the problems encountered seven papers were presented at a session devoted to this general subject.

The first of these papers, "The Function of Fly-Wheels in Electrically Operated Rolling Mills," by Mr. H. C. Specht, of Pittsburg, presented some specific cases of designing fly-wheel capacity to give, as near as possible with a rolling-mill motor, a constant motor load equal to the average of all the loads on the rolling-mill driven by that motor.

In the paper by Mr. E. W. Yearsley, of the Midvale Steel Co., Nicetown, Pa., "Rolling-Mill Motors," the practical requirements of steel-mill motors were cited. The control apparatus was held to be the weak part of the electrical system at present. The author believed that more tests on present installations were advisable to obtain a better knowledge of power and speed requirements of the rolling operations, so that fly wheels could be better designed to wipe out sharp peak loads.

In the paper of Mr. E. Friedlander, of the Carnegie Steel Co., Braddock, Pa., "Electric Driven Rolling Mills," the author called to mind the better knowledge now possessed of the power required during the different stages and under different heat conditions. This was due to the comparative ease of determining energy input into the mill motors as compared with steam drive.

Mr. R. Tschentscher, of the Indiana Steel Co., South Chicago, Ill., recited the development of electric drive for steel mills in his paper, "Electric-Power Problems in Steel Mills." The mills were divided into four classes: (1) rolling plants where billets were purchased and rolled out, power-plant capacity about 200 KW., (2) manufacturing mills, buying iron and scrap, operating open-hearth furnaces and rolling mills, having an electric power-plant capacity between 300 and 1,500 KW., (3) self-contained plants buying ore flux, etc., and running blast furnaces and all steel machinery, having a power-plant capacity of 500 KW. upward to several thousand, (4) the self-contained plant supplying power for light, etc., outside the steel industry, having a power-plant capacity of no less than 1,000 KW. and as high as 2,000,000.

The South Chicago works of the Illinois Steel Co. were taken as typical of class 4. This had evolved from one having 120 KW. generator capacity to 26,000 KW. in the power plant. There was 75,000 HP. of motors in the works. A plant which did not have to pass through such an evolution would be the new type of class 4. The steel output might be 2,000,000 tons per year and the power plant might develop over 1,000,000 KW. from the waste gases of blast furnaces and coke ovens without burning any coal under boilers for power.

Mr. J. L. Woodbridge, Chief Engineer of the Electric Storage Battery Co., Philadelphia, in the paper, "The Application of Storage Batteries to the Regulation of Alternating-Current Load at the Plant of the Indiana Steel Co., Gary, Ind.," gave a brief description of the battery plant, showed a few of the unusual features and gave an account of the commercial operation.

Briefly told, the battery consisted of two series of 125 cells, operating at a maximum of 225 volts and having a total capacity of 1,089,600 ampere-hours. These are installed in a two-story building ventilated with filtered air. The transfer of energy between the battery and the alternating-current circuit is through rotary converters and special regulating exciters. The so-called "split-pole converters" were used so that the direct-current voltage might be given a great range without change of the alternating-current voltage impressed on the converter. In these machines the field coils are subdivided and, the sub-poles being separately excited, the character of the field magnetization can be changed independently of the impressed alternating current. This allows independent change of direct-current voltage. In the Gary installation the changes of the fields of the split-pole converters are automatically made by the changes of the alternating-current load so that during the peaks the battery drives the converter and repays alternating current.

The arrangement of the machine which excites the regulating fields of the "split-pole" converter to make this possible is interesting. The armature of this regulating exciter has both commutator and collector rings. The latter, six in number, have brushes bearing on them and connecting to three current transformers in the circuit whose alternating-current load is to be regulated. Thus the current fed into the armature, producing an electrically rotating field, depends on the load carried by these circuits. A synchronous motor revolves this armature against the direction of the electrically revolving magnetic poles so that the latter stand stationary in space. The flux thus made stationary

is neutralized by the ordinary field windings of the exciting regulator. Therefore if the stationary flux resulting from the alternating-current connection varies in magnitude there is an unbalance of the fields, voltage is generated at the brushes and current is led off to the split poles of the converter. Thus the fields of the converters (and the direct-current voltage) are changed by a change in load on the alternating current-circuits and the storage battery either repays or draws current as may be necessary to wipe out peaks or depressions of the main load.

Concerning the operating results Mr. Woodbridge had this to say:

> The effect of the battery regulation on the operation of the entire plant is very marked. In the first place the battery permits the shutting down of one 2,000-KW. gas-engine unit, which would be required if the battery were not in service. Even with this additional unit in service, the operation of the plant without the battery involves constant attention by the switchboard attendants and a continual adjustment of generator field rheostats and engine governors. Notwithstanding this constant attention, it is impossible without the battery in service to maintain satisfactory voltage regulation, and the speed of the engines is continually varying over a wide range, whereas with the battery in service the regulation of speed and voltage is all that could be desired.
>
> One of the most obvious of the practical commercial results of the battery regulation is the increased speed at which the rolling mill may be operated. When the battery is not in service a very appreciable reduction in the daily output is observed due to irregularity of speed and voltage. In addition to this regulating effect, the battery is an ever present source of energy to supply power in case of some emergency such as the sudden shutting down of one of the prime movers or any other partial interruption of the power supply.

A paper, "Power Requirements for Rolling High-Carbon Steel of Small Section," by Mr. Brent Wiley, of Pittsburg, was largely a collection of load curves taken from a test of a 9-in. merchant-bar mill driven by a 600-HP., 2,200-volt induction motor. The product was a high-carbon steel of small section. There were also two more papers, "Automatic Motor Control," by Mr. H. E. White, of Schenectady, and "Electric Control of Rolling-Mill Motors," by Mr. C. T. Henderson, of the Cutler-Hammer Co., Milwaukee, both dealing largely with the design of automatic controllers with electrically-operated contactors to bring mill motors to proper speed, when once the starting process is begun, in minimum time and with maximum safety.

ENGINEERING EDUCATION.—While there were only two papers on this general topic they sufficed to start a discussion that lasted nearly two hours.

"The Training of Non Technical Men," by Mr. C. R. Dooley, of Pittsburg, gave an account of the facilities in and about the Westinghouse works for a sort of training to make shop men good general mechanics rather than machine specialists. This was by shop-apprentice courses where the boys were given theoretical instruction during their employer's time and as a part of their apprenticeship. For the many other men, largely intermediate between the engineering force and the shop mechanics, an evening school had been organized, but not under the direction of the Westinghouse company. Here the more ambitious mechanics, salesmen or draftsmen could get training in mathematics, mechanics, chemistry, physics, drawing, shop practice, steam engineering, English, etc.

This school, it was said, has a remarkable spirit, and maintains perfect self organization and discipline. The graduation period is marked by the same evidences of enthusiasm as is any good school. The social life of the students has developed until there is a glee club, a mandolin club, basket-ball team and a baseball team. A monthly bulletin is also edited and published. The total enrollment is in the vicinity of 300 and 80% of those entering graduate.

In "The Value of the Classics in Engineering Education," Dr. C. P. Steinmetz, speaking in his capacity as an educator, held up the Latin and Greek languages and literature as the most valuable part of an engineer's education aside from those studies that allowed him to earn a mere living. These classics he claimed opened to the student worlds of art, literature, organization and citizenship far different from those of today. The modern languages he claimed were not in the same class with the old languages as they could open no fields of thought appreciably different to that opened by the student's own tongue. The present method of teaching the classics was not approved, however.

The general trend of the long discussion was that breadth of training was essential for the leaders in engineering fields and desirable for all the mass of industrial workers. The function of any education, it seemed to be agreed, was first for self-support and secondly toward the support of civic institutions. Various classes of workers were held to be necessary, ranging from those whose occupation was largely manual and only slightly intellectual to those whose activities were largely mental and only slightly manual. The higher one went in the ratio of mental to manual effort, in general, the more essential was breadth of training. There was a general consensus of opinion that the breadth of view Dr. Steinmetz sought, through the study of Latin and Greek, could be gained by the study of the history of nations and of institutions like the common law, by the study of the elements of sociology, economics and of the literature of all nations—not merely of the Greek and Latin.

MACHINE DESIGN.—"An Alternator for 100,000 Cycles" was the title of a paper by Mr. E. F. W. Alexanderson, of Schenectady, describing a type of machine developed primarily for wireless telegraphy and telephony. This generator is an inductor type of alternator, in which neither field nor armature windings are revolved, but the change of magnetic flux is made by revolving a laminated and toothed iron disc past the slotted faces of a field in which slots the armature wires are placed. This disc is carried on a tapering shaft which has sufficient flexibility to allow the disc and shaft to rotate about its mass center as in the De Laval steam-turbine rotor. Normal operation of this generator is at 20,000 r.p.m. and at 1,700 and at 9,000 r.p.m. the rotor is subject to severe vibrations. At these critical speeds the natural vibration of the shaft and disc in the bearings coincides with the speed of revolution. Driving was at first by means of a belt, but it was found more practical to use gears. In addition to the ordinary bearings a second set is added, one on each side of the disc to damp the vibrations at the critical speeds and to help resist the thrust of magnetic unbalance.

In running such a generator, a condenser having sufficient electrostatic capacity to balance the inductance of the windings, is placed in series with the machine to give maximum output. This alternator has in every way the general characteristics of an ordinary machine.

Several papers on design were presented of greatest interest to specialists and these are listed below.

"Designing Heavy Capacity Fuses," L. W. Downs, of Providence, R. I.

"Comparative Costs of 25-Cycle and 60-Cycle Alternators," C. J. Fechheimer, of Milwaukee.

"Calculation of Iron Losses in Dynamo-Electric Machinery," I. E. Hanssen, of Pittsburg.

"Auxiliary Poles for Direct-Current Machines," J. N. Dodd, of New York City.

"Electric Measurements on Circuits Requiring Current and Potential Transformers," L. T. Robinson, of Schenectady.

"E. F. M. Wave Shape in Alternators," by Prof. C. A. Adams, of Boston.

"Pole-Face Losses," Prof. C. A. Adams, Messrs. A. C. Lanier, C. C. Pope and C. O. Schooley, of Boston.

THE LAGUNA DAM, recently completed by the U. S. Reclamation Service at Yuma, Ariz., safely passed a flood of 139,000 sec.-ft. in the latter part of June. It is reported that there was no damage either to the dam or to the miles of levees which protect the bottom lands below the dam. An article describing the work of completing this dam by closing the gap left for the passage of the stream during construction was published in our issue of June 10, 1909. Earlier articles appeared Feb. 9, 1905, and Feb. 27, 1908.

New San Francisco Shipping Piers, with Concrete Cylinder Foundations and Fireproof Deck and Superstructure.

By JOHN G. LITTLE.*

The Board of State Harbor Commissioners for San Francisco harbor has adopted an elaborate scheme of harbor improvements which it is now carrying out. A very advanced type of pier construction was adopted for the group of new shipping piers under construction at the foot of Townsend Street as part of the scheme. These piers are of thoroughly fireproof construction, having no wood or other perishable material in either substructure or superstructure except fender and mooring piles, and window frames and louvers.

Their noteworthy feature is the steel-and-concrete substructure. It consists of reinforced-concrete cylinders penetrating to hard bottom, with a conical spread at the lower end to secure the desired bearing area. The deck supported by these cylinders has steel beam framing and a reinforced-concrete slab filling.

The superstructure consists of steel side framing and roof trusses, with reinforced-concrete walls and roof, and Kinnear metal rolling doors.

Typical drawings of Pier 38, now under construction, are shown herewith (Figs. 1 to 4). They exemplify characteristic features of the construction used.

The concrete pillars are an evolution from the concrete-sheathed wooden pile construction used in San Francisco piers during the past eight or ten years. The teredo is very active in the waters of the Bay of San Francisco, the life of plain wood piles being scarcely twelve months, and the average life of creosoted piles about nine years. Nearly ten years ago, therefore, the Harbor Commissioners adopted a form of concrete-protected pile construction. Ordinary wooden piles, driven to secure the required bearing power, were provided with a concrete casing molded in a cylindrical wooden form dropped over the pile after driving. It was considered that the layer of concrete acted also to stiffen the pile against buckling, and added somewhat to the bearing power through the bearing of the lower end of the concrete on the soft soil of the bottom.

The present condition of piers built on this type of substructure is not very encouraging. The wooden forms being left in place, the concrete was protected for a considerable period of time. After the activity of the teredo had destroyed the form, however, the concrete was exposed; in some cases it has been far from perfect, and any break in the concrete casing allows the entrance of the teredo into the pile, which of course means its destruction in a short time.

The existence of a hard stratum of clay about 15 ft. below the mud line (see lines of strata in the elevation, Fig. 2) suggested the use of solid reinforced-concrete cylinders bearing directly on the hard bottom. The concrete cylinder construction is simpler than the concrete-protected pile construction, and has the great advantage that any desired load capacity can be secured by proportioning the cylinder to suit and by embedding steel reinforcement in the concrete.

It happens in the case of these San Francisco piers that the diameter of pillar required to make a shaft of suitable strength is less than the diameter needed to give a sufficient bearing area on the bottom. On this account the pillar is belled out by a suitably shaped cast-iron form attached to the lower end of the wooden form cylinder of the shaft.

The fully fireproof deck is another innovation in these piers. With the old types of pier construction, in some cases the floor was constructed of steel girders protected by concrete covering supporting wooden floor joists, though all-timber decking was more often used. With the reinforced-concrete pillars, however, it is an easy matter to provide a complete fireproof floor system, using either steel or reinforced-

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concrete beams and a concrete or reinforced-concrete floor filling. In this case steel beams protected by a good thickness of concrete form the main members of the floor, and the filling is a reinforced-concrete slab 6 ins. thick. The plans and sections in Figs. 1 and 3 represent this arrangement clearly. The beam protection is so dimensioned as to give a minimum thickness of 2 ins. of concrete over the extreme corners of the flanges.

The floor construction is designed for a live-load of 500 lbs. per sq. ft., and a 75-ton locomotive on the railway track.

The paving of the deck consists of asphalt on the larger part of the surface, and vitrified brick on the driveway at the center. The superstructure is a simple column-and-truss bent construction. Its bents are spaced 30 ft. apart longitudinally and are braced together by eave struts and intermediate longitudinal struts, besides the roof purlins and diagonal bracing in the roof plane. The concrete walls are 6 ins. thick. The roof concrete, also a reinforced slab (2¼ ins. thick) is covered with felt-and-asphalt roofing.

REINFORCED-CONCRETE CYLINDERS.—Borings on the site of Pier 38 showed a hard stratum of clay about 40 ft. below low water, which it was judged was capable of carrying safely a superimposed load of 5 tons per sq. ft. This requires ordinarily a larger footing area than that furnished by the 3½-ft. diameter of the shaft of the pillars. By flaring out the base of the pillars, any desired increase in bearing area is obtainable, and each footing may therefore be proportioned to its loading, according to its position in the structure, without altering the size of the shaft.

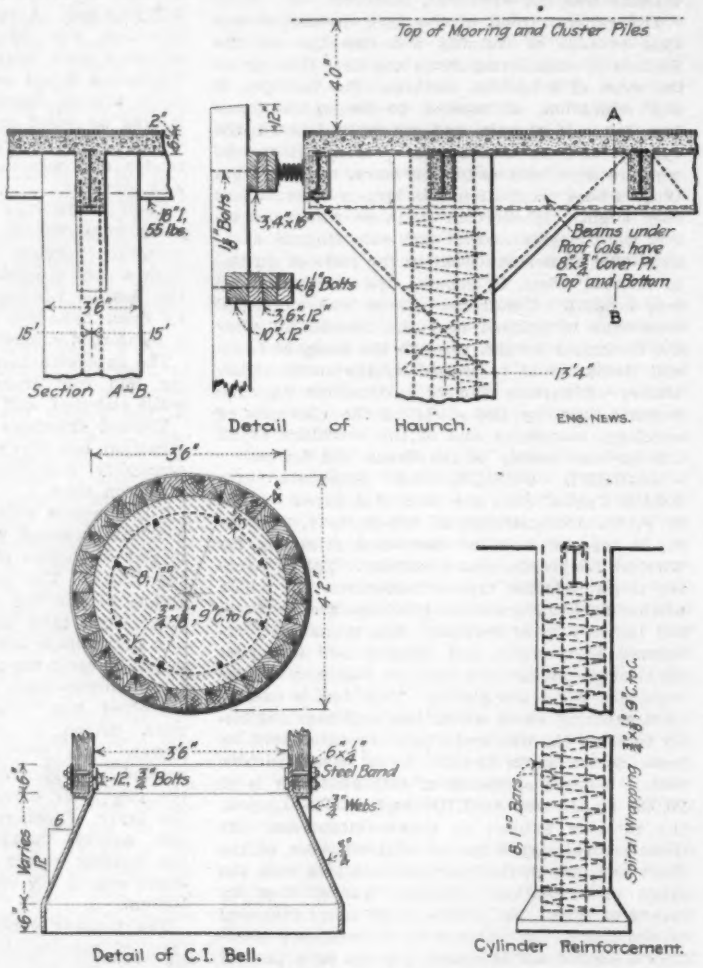

FIG. 1. DETAILS OF REINFORCED-CONCRETE CYLINDER, FORM AND FLOOR CONNECTION.

The form cylinders are built up of staves 4 ins. thick, planed from 4 × 4-in. yellow fir lumber and banded with flat iron. The cast-iron bells at the bottom are bolted to them. The assembled form is jetted down into place, assisted by a hammer where necessary.

Four large water jets in each cylinder kept playing on the bottom, loosen up the soil thoroughly enough so that the cylinder sinks with little assistance until the top of the clay foundation stratum is reached. Here some hammering is generally required to secure the desired 12 ins. of penetration. A thorough sealing of the bottom of the form in the clay is demanded by the requirement that it must be possible to pump out the forms and work entirely in the dry. In fact, it is desirable to obtain a sufficiently thorough seal that the bottom can be inspected as to its character of soil before placing contract.

The specifications for this part of the work may be of interest:

> The staves for cylinders must not be less than 4 × 4-in. merchantable yellow fir, and planed so that the same, when bound together, shall form a perfect cylinder; staves must be free from rotten knots or such as are liable to drop out in driving and any defects that may prevent the making of water-tight joints. One splice only will be allowed in each stave, it being understood that the length of a stave is the length of the cylinder. Splices in staves must be made and located as directed by the Assistant State Engineer. Each cylinder must have a bottom hoop of not less than ¼ × 6-in. iron riveted to each stave; the remainder of the hoops must be made of not less than 5/16 × 2-in. band iron and they must be spaced not more than 2 ft. c. to c. The latter hoops to have an E. C. Tecktonius tightening lug or its equal.
>
> The contractor may at his option substitute for the wooden stave cylinders, concrete cylinders having an outside diameter equal to the inner diameter given for wooden stave cylinders, and provided with concrete bells of outer dimensions equal to the inner dimensions given for cast-iron bells on wooden stave cylinders. The concrete cylinders must be reinforced with at least equal reinforcement for concrete in wooden stave cylinders. * * *
>
> The cylinders must be set in the exact location shown on plans and driven or jetted down until their feet rest on the hard bottom.
>
> Care must be taken in sinking the cylinders to injure the heads of same as little as possible. The contractor must use special frames fitting over the heads of cylinders if same are driven, and the same must be approved by the Assistant State Engineer.
>
> All water and soft mud in all cylinders must be removed from said cylinders to a depth at least level with the "Hard Bottom" line, shown on profile, to secure proper conditions for the formation of perfect concrete. After the cylinders have thus been relieved of water and soft mud, they must be allowed to stand empty a sufficient time to demonstrate their ability not only to prevent the resurgence of water and soft mud, but also to withstand leakage and collapse at high water. * * * [Then] the contractor shall fill them with loose concrete to a depth of at least 6 ins.
>
> The contractor must then set in each cylinder, the steel cages of rods and bands or wire mesh as shown on drawings, placed as a reinforcement of the concrete in cylinders. These rods or wire mesh cylinders must be set concentrically with the cylinders as directed by the Assistant State Engineer. The contractor must then place and carefully tamp concrete around same and must continuously prosecute said operation until the concrete shall have been brought up to at least 1 ft. above the level of high water then prevailing. The remainder of the cylinder shall be filled within 24 hours thereafter or as soon as the Assistant State Engineer may direct. At no stage of the process of filling the cylinders must there be any surplusage of water appearing on the top of the concrete. All cylinders must be kept plumb until finally secured in permanent position.

The concrete for these pillars is a 1:2:4 mixture, made with stone ranging between ½-in. and 1½ ins. It is specified that batch mixers must be used. The reinforcing metal, when consisting of plain bars, band iron or wire cloth, is specified to be of the same quality as the structural steel, for which the ultimate strength is fixed at 60,000 to 70,000 lbs. per sq. in., with an elastic limit not less than one-half the ultimate strength. When deformed bars are used, they are required to have an elastic limit not less than 54,000 lbs. per sq. in., and to be capable of bending cold 180° around a diameter equal to twice the thickness of the bar without sign of fracture on the outside of the bend.

The concrete of the floor slab of the deck is required to be monolithic with the concrete covering of the beams and the concrete of the top portions of the cylinders. This also is 1:2:4 concrete, as is also the concrete of side walls and roof. For the walls a brush coating of pure cement grout is specified.

FENDER TIMBERING.—An interesting feature of the construction is the fender timbering on sides and ends. Some details of this are shown in Figs. 1 and 3. Along the sides a row of wooden fender piles spaced 3 ft. on centers is driven, about 2 ft. from the edge of the concrete floor slab. Two tiers of timber longitudinals are fastened to the pier side of these piles, one just below the level of the deck, the other some 5 ft. lower down. The upper longitudinal is cushioned against the pier by railway-car springs spaced about 15 ft. along the pier, these forming the only connection between the fender system and the pier itself. The fender system along the end of the pier is similar, but has no springs and only one longitudinal. It is connected to the pier by bolts and timber blocking at intervals of about 3 ft. Outside of the fender longitudinals at the corners of the pier there is a heavy cluster of piles, driven close and bolted together in all directions and to the fender timber by 1⅛-in. bolts. Above the pier floor the cluster is required to be blocked in solid, and directly above the deck the cluster is bound together by six turns of 1-in. galvanized wire rope, drawn tight by means of thimbles spliced into the ends and seized with galvanized wire.

Fender piles and cluster piles are of Douglas yellow fir not creosoted. They are required to be not less than 16 ins. in diameter inside of bark at the butt, and must be sized to 15 ins. where the fender longitudinals attach. The cluster piles are cut off 4 ft. above the deck, the fender piles 12 ins. above the deck.

MOORING PILES.—Mooring piles are provided at intervals along sides and end of pier. These pass through holes in the deck 2 ft. back from the edge of the concrete. A collar, made of a 6-in. 13-lb. channel formed into a ring 20 ins. inside diameter, is molded in the concrete deck at the location of the mooring pile and tied back by 1-in. bars embedded in the floor concrete and running back a full panel (see Fig. 3). After the mooring pile is driven through this ring, the space between pile and ring is filled with wooden blocking, wedged tight. These mooring piles are creosoted.

The work was designed and is being supervised by Mr. Ralph Barker, Assistant State Engineer, under the direction of Mr. Nathaniel Ellery, State Engineer, for the Board of State Harbor Commissioners of California, W. V. Stafford, President. Pier 38 is being built by the Healy-Tibbits Construction Co., of San Francisco, and Pier 40 by the Wakefield-Pacific Co., of San Francisco and Portland. The contract for the construction of Pier 36 has just been let to the Associated Contracting Co., of San Francisco.

Pier 40 covers an area of 130 ft. × 650 ft.; Pier 38 is approximately the same size. Pier 36 is irregular in plan and contains a ferry slip for freight-cars, and is in many ways different from the other two, although the same general scheme of construction has been followed.

The contract price for Pier 40 complete was $304,000, for Pier 38 $279,000, and for Pier 36 $364,000, the cement in all cases being furnished by the State free of charge to the contractor.

"Hydroplanes" or "Skimming" Boats.*

Vessels which greatly reduce their displacement when traveling at high speeds are generally called "hydroplanes," but this name is not altogether satisfactory, as the surfaces on which they glide are not always planes. To call such vessels "gliders" or "skimmers" has been suggested as more appropriate, but the former title suggests smooth motion, and this is not always a characteristic of their behavior. In this paper, therefore, the latter word will be used to describe boats which at high speeds are heavier than the water they displace.

"Skimmers" are very ancient, and are still used by some of the inhabitants of islands in the Pacific, in what must be their simplest form. A single slab of wood, rounded at the edges, is employed, and supports a man where huge waves form a sufficient slope to enable him, assisted by gravity, to attain the necessary start. After the start the man will even assume a standing attitude on the board which skims along before the advancing wave.

The problem of making a boat glide steadily on the surface of the water is not an easy one. Steadiness can probably be gained by using a number of planes, but this is likely to increase the frictional resistance. When a number of planes are used to support a given load, each must be of less length than when only one is used. The friction per unit of surface being greater for short surfaces, it is improbable that a smaller total surface will be sufficient, and the necessary power required for a given speed must be greater. Mr. Froude's opinion was that the best results could be obtained from a single plane, but this must maintain a particular angle to the water surface. About the year 1872 he made a model, in which three surfaces were attached to a frame, and towed in such a position that the wake of either of the three did not interfere with the water on which any one of them had afterwards to run. Mr. Froude proved by theory that the angle the plane should have to the line of motion should be such that the resistance due to surface friction should be equal to the resistance due to gravity, or the horizontal component necessary to balance the weight of the vessel on the incline of the supporting surface. This inclination he found to be about 1 in 17, so that the total resistance amounts to about 1/8.5 of the weight or displacement of the vessel. This friction, however, depends on the value to be attached to the surface friction, and this also varies with the quality of the surface and the length of the surface rubbing. This inclination is therefore not a definite angle.

Mr. Froude found, when experimenting with the "Ramus" model that the front plane lifted entirely above the water surface as the speed was increased, and the center of gravity apparently overhung all natural support. It is evident that this effect can only be produced by the pressure on part of the surface being less than that of the atmosphere.

The author had made some experiments to illustrate this with a model, the bottom of which was, for the most part, a simple plane, but at the after end the surface could be turned to a slight angle, and the result was that in some cases the model even jumped clear of the water surface. When the bottom surface was left flat throughout its length, the model glided smoothly over the water; but when the tail part was bent down, the model promptly dived. These experiments seemed to show that the endwise vertical section of the bottom of a skimmer should be a straight line; although a hollow curve would seem to promise a more even distribution of pressure on the bottom.

In a boat intended for skimming or sliding on the surface of the water with the least possible disturbance, there are a number of elements to consider. Unfortunately, these do not all lead to the same proportions of design. In the first place, the lifting force depends on the amount of surface and on the speed; while the friction for a certain amount of surface will decrease with greater length. The speed at which skimming will commence, however, increases with the length. This limit should naturally be kept as low as possible. Below a certain velocity, the performance of a skimmer model is very bad, owing to the formation of large waves, which allow the stern of the vessel to fall and greatly increase the angle of the planes, thus rendering it more difficult for the vessel to mount to the surface and skim. This difficulty may be lessened by (1) extending the amount of supporting surface, or (2) reducing the weight of the vessel, the surface remaining the same. The great strength necessary to withstand the shocks to which this kind of boat is subjected, renders reduction of weight difficult. A boat very wide and short in shape leads to excessive air resistance, and this resistance becomes quite an important factor at speeds of about thirty miles an hour. For this reason too great a width must be avoided.

The wave formation, when a skimmer is moving below the skimming phase, resembles that of an ordinary vessel, but the waves are larger in proportion to the size of the vessel, and diverge at a wide angle.

Skimmers are near relations to flying-machines, and although smooth water would appear to form a definite plane on which to travel, a boat of this kind, when moving at high speed tends to oscillate vertically and jump from the water surface, and under some conditions to dive below. The skipping action has been already referred to above, the author's model having been frequently made to leave the water temporarily in this way, at relatively high speed.

The position of the center of gravity is very important to get the best result. In the models he had tested, improvement appeared to take place as this was moved aft up to the stage when skipping or flapping commences. This dancing motion, if allowed to develop, may become dangerous, but the best results seem to coincide with its commencement.

The growth of resistance in a skimmer differs from that in an ordinary boat in a very marked manner: it rises very rapidly at first with increase of speed, but when the condition of skimming is established, it may even fall temporarily, and afterwards only rise very slowly, so that the power required only increases a little faster than the velocity. The boat form is much the better for lower speeds, while above the point where the resistances are equal the advantage is greatly in favor of the skimming model.

It is often supposed that air passes under skimmers, but this is only likely to take place when the water sur-

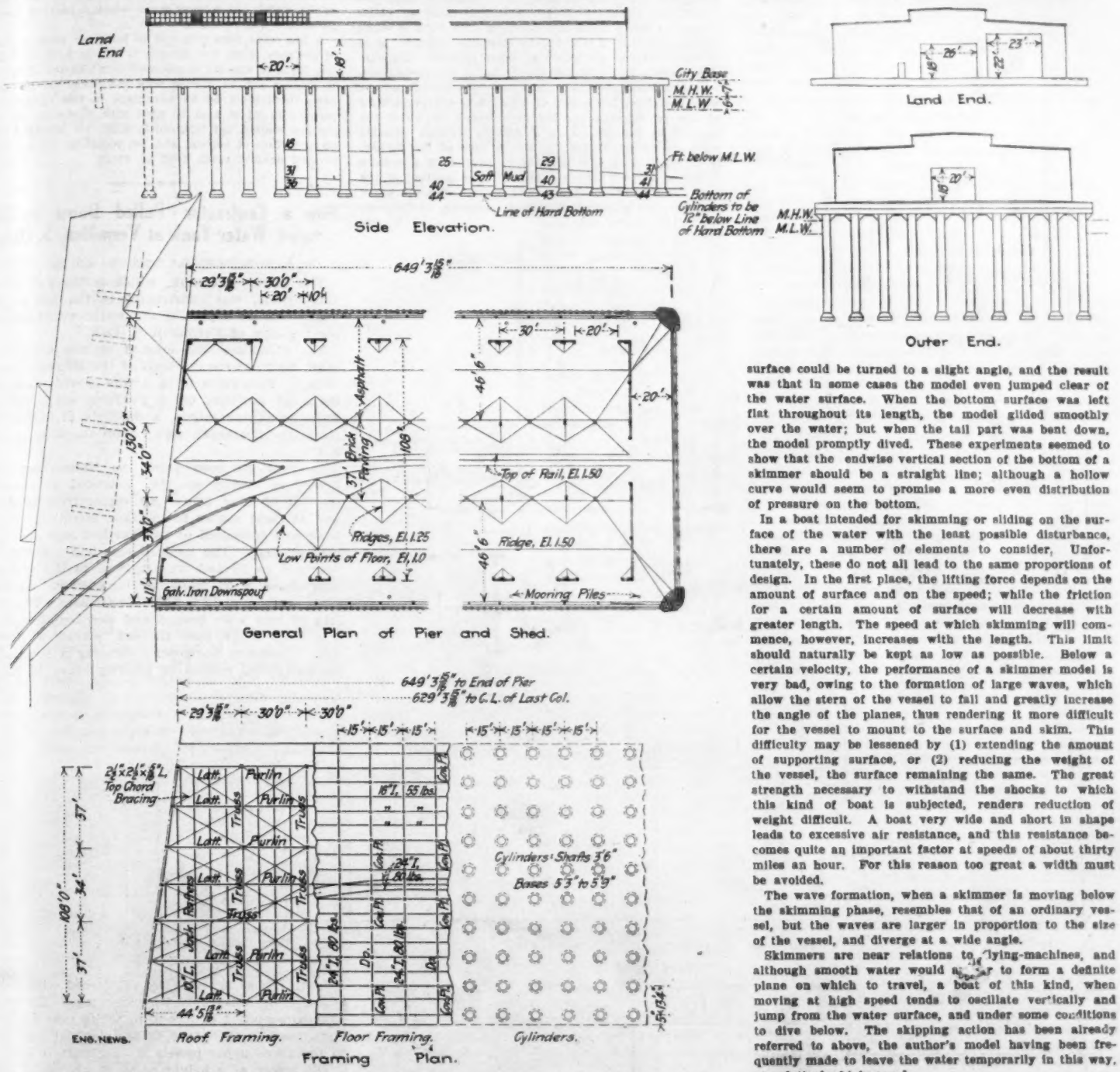

FIG. 2. PART FLOOR PLAN, ROOF PLAN AND ELEVATIONS OF PIER 38, SAN FRANCISCO HARBOR.

*A paper read by Sir John I. Thornycroft, F. R. S., at a meeting of the Motor Yacht Club, March 4, 1909, from "Engineering," London, March 12.

face is broken. It is well known that a jet of water impinging on a surface, even at an acute angle, does not all pass under in the direction of the jet. A small part at a point near to the surface has its motion reversed. This reversal renders the passage of any air between the jet and the surface impossible. If, however, the surface of the moving water is broken or churned into foam, then that mixture of air and water will pass along the surface. What will be the effect of this seems uncertain, but the late Lord Kelvin was thoroughly of opinion that the friction of this mixture would be greater than that of solid water. The form of boat used by M. Fauber is adapted to eject any air from under it, and, perhaps, from this some advantage was obtained which tended to balance the loss which might be expected to arise from the use of so many short skimming surfaces.

The author also tried models intended to carry air under them in a hollow. This construction is favorable to high speed, for the angle of the surface rubbing on the water can be considerably reduced by the adoption of this plan, so reducing the resistance due to gravity on the incline. At the same time the friction due to air on the supporting water is much less than that due to water on a solid; and if the air could be carried with the model, the power required to force it under would be small and would only amount to that necessary to make up waste. It may be safely taken that when air goes under a vessel without forcing, it will fail to give any support. It is evident that the air under a vessel to serve this purpose must have a pressure above that of the atmosphere.

In a skimmer in which the water rubs directly on the bottom surface, the center of pressure is always forward of the center of area; but in a vessel in which the support is due to air in a hollow, it must coincide with the center of gravity of the area of the hollow. It is much more advantageous to have such a definite center of support than the ever-shifting position of the center of the lifting force due to contact with the water surface. The practical difficulties, however, to be overcome in producing a type embodying this principle are many. Care must be taken, for instance, to avoid disturbing the water surface by the surfaces which must surround the hollow containing the air. If this be not done, waves will form and increase the rubbing surface, besides taking up energy in their formation. With a model the author designed to overcome these difficulties the resistance had not proved very small, but it was observed that the wake of the model indicated very little disturbance of the water surface. This model was very wide and nearly circular in plan, and it skimmed at quite a low velocity. It may be interesting to state that this form of model was made in 1873. A similar model was towed from a launch at about 8 knots; this speed was more than sufficient to make it rise to the surface. When air was forced under it, the resistance was greatly reduced, but it was never very low. At the best it fell to a value of about one-quarter the weight of the model. At a lower speed—about 5 knots—a greater resistance was recorded.

It has often been proposed to force air under a vessel of ordinary form, and Ericson is said to have tried this, but found it was not a success. In a skimmer, however, if the air can be carried with the model for the most part, there must be an advantage in this system, and perhaps it might also be used with advantage in very shallow vessels, not intended to skim, but having a very large surface of bottom, and not requiring the air to be pumped against much head of water.

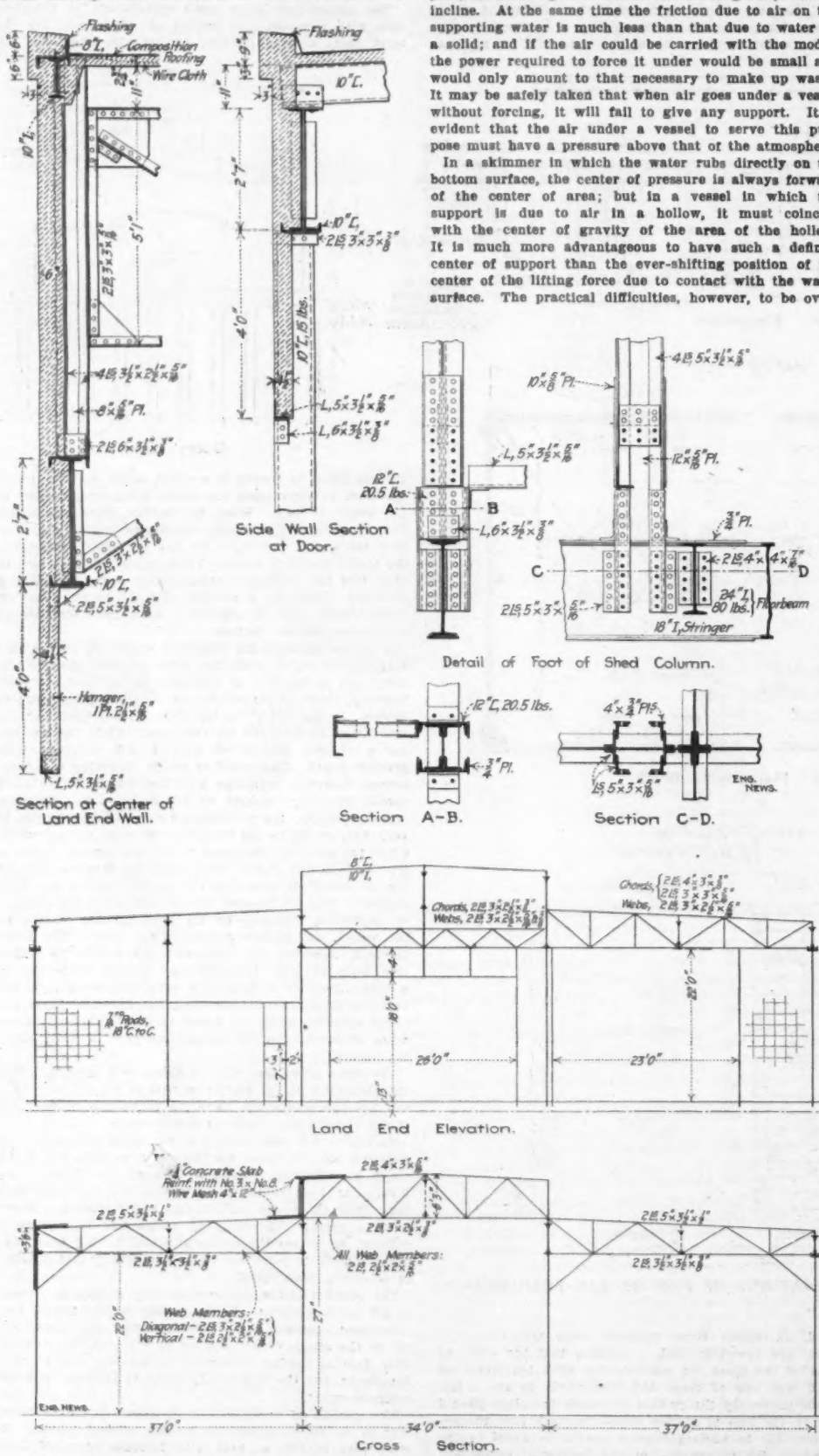
Flashing
Composition Roofing
Wire Cloth
Side Wall Section at Door.
Detail of Foot of Shed Column.
Section at Center of Land End Wall.
Section A-B.
Section C-D.
Land End Elevation.
Part Side Elevation.
Cross Section.
Web Members:
Diagonal - 2 L, 3" x 2½" x 5/16"
Vertical - 2 L, 2½" x 2" x ⅜"
All Web Members:
2 L, 2½" x 2" x ⅜"
30' C. to C. of Cols.
ENG. NEWS.

FIG. 4. STEEL FRAME OF SUPERSTRUCTURE FOR SAN FRANCISCO PIERS.

How a Contractor Pulled Down an Elevated Water Tank at Vermilion, S. Dak.

By A. B. McDANIEL,* Assoc. M. Am. Soc. C. E.

The tower and tank, which is the subject of this article, was constructed in the fall of the year 1891, as a part of the water-works system for the city of Vermilion, S. Dak.

The structure was erected on the edge of a bluff, 80 ft. above the level of the Missouri River valley. Two years later it was moved entire, in its erect position, on a platform supported by large iron pipe rollers, a distance of about 300 ft. to the northeast, away from the edge of the bluff.

As may be seen from the illustration, the tower, as finally erected, consisted of wooden trestles of four bents, set respectively parallel and at right angles to the true meridian. Each bent was composed of two vertical legs and two inclined legs. The inclined legs had a batter of nearly 1 in 10 and were spliced to the vertical legs about 8 ft. from the tank floor. All the posts were solid 12 × 12-in. fir timbers. The two sets of legs were braced and tied together with 4 × 6-in. white pine timbers, placed with the 6-in. dimension horizontal, forming a rectangular horizontal system of bracing every 10 ft. in height. These braces were reinforced with ¾-in. round iron rods, bolted to the legs. Diagonal braces composed of 3 × 9-in. white pine timbers were used in the plane of the vertical legs and in the three upper panels of the inclined legs.

The tower, at a height of 80 ft. above the tops of the foundations, supported a frame platform of simple construction. On this rested the tank, composed of wooden staves bound with adjustable iron hoops. This tank was 24 ft. in diameter and 16 ft. high, and had a capacity of about 54,000 gals. Ordinarily, the tank was kept about half full of water.

The whole structure was supported on masonry piers, composed of large blocks of quartzite laid in cement. These piers were 18 ins. square on top, 6 ft. high and had a bearing area of 18 sq. ft.

The 8-in. wrought-iron riser pipe was cased in with a simple boxing of tongued-and-grooved white pine boards 2 ft. square.

*Professor of Civil Engineering, University of South Dakota, Vermilion, S. Dak.

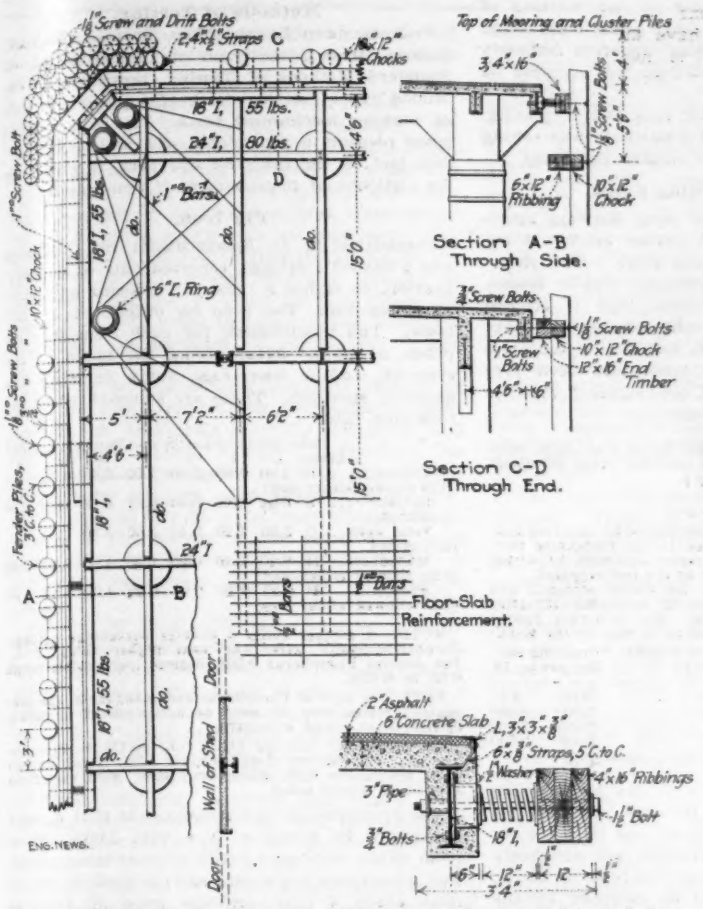

FIG. 3. DETAILS OF FLOOR, FENDER TIMBERING AND ANCHORAGE OF MOORING PILES, SAN FRANCISCO PIERS.

About five years ago, the tower was observed to move slightly on the tops of the piers. This movement was accompanied by a slight buckling of the riser pipe at about one-half its height. The movement of the posts gradually increased, and, about 18 months ago, pieces were bolted to the sides of the posts at their lower ends so as to distribute the unequal loading more uniformly over the piers. A ½-in. circular iron rod, made in loop-ended sections 3 ft. in length, was placed around the riser pipe at about one-half its height, and the ends of the rod were securely anchored to two large trees, about 32 ft. north of the tower.

On May 1 of the present year, the structure had settled until the riser pipe was nearly 7 ins. out of plumb and the posts of the south and east bents were deflected to the south and west about 9 ins. Many of the braces had fallen to the ground and the whole structure was somewhat twisted and deflected out of shape. The tank was leaking badly. On May 7, the water company became alarmed at its condition and ordered the structure taken down.

A thorough search in the city and neighboring country disclosed the fact that no dynamite or similar explosive could be obtained at short notice. It became evident that other means of demolition must be employed. A 16 × 16-in. timber, 24 ft. in length, was placed across the top of the piers supporting the inclined posts of the east bent and diagonally between and against the lower ends of these posts. One end of the timber was made fast to the north post and a block and tackle was fastened with a chain to the other end of the timber, projecting 15 ft. beyond the south posts. A 2-in. hemp rope, one of whose ends was fastened to a large tree nearby, was run a distance of 200 ft. from the timber, where twenty men soon wrenched the ends of the post away from the piers. The same lever was used to remove the lower ends of the vertical posts of the same bent from their supporting piers. As soon as the latter posts were displaced, the posts of the west bent began to rise very slowly and in about three minutes the structure fell.

As may be seen from the illustration, the structure fell in a general southwesterly direction, while the tank fell in a nearly vertical line. The plane of fracture was about one-half the height of the tower, at the point where the posts were spliced. The posts and braces, although merely coated with a common mineral paint of cheap grade, were found to be well preserved. The lower part of the tank and the supporting platform were in a badly rotted condition, due to leakage.

The work of demolition was done by Mr. Eric Nylan, Contractor, under the general supervision of the writer.

Pulling Down an Elevated Water Tank at Vermilion, S. Dak.

(Photograph copyrighted by W. H. Lawton, Vermilion, S. Dak.)

RAISING A RAILWAY BRIDGE by wrecking cranes to allow steamers to pass was performed by the Chicago, Burlington & Quincy Ry. a few weeks ago to permit two steamers to pass from the Wisconsin River to the Mississippi River near Prairie du Chien, Wis. There is no navigation on the Wisconsin River, but the two steamers were sent down to enter service on another route. The railway crosses the river near its mouth, and a 65-ft. span was raised about 6 ft. to clear the steamers' smokestacks. Two high capacity wrecking cranes with long booms were used. Work of this kind has been performed in several instances with small bridges over drainage ditches in order to allow the passage of dredges. An instance of this kind was described and illustrated in Engineering News of April 11, 1907, p. 391.

Annual Meeting of the American Society for Testing Materials.

The 1909 meeting of the American Society for Testing Materials, held at Atlantic City, N. J., June 28-July 3, enjoyed very favorable conditions, among them a large attendance (over 250) and excellent weather. It was distinguished also by a departure from the practice of holding simultaneous sessions of different sections, which practice grew up some four years ago when the volume of proceedings became so great as to make it impossible to crowd the meeting into three days without some such device. The practice had caused dissatisfaction on the part of those who desired to attend, for example, both the Iron & Steel and the Cement & Concrete sessions. Therefore only single sessions were held this year, nine in all, extending from Tuesday noon, June 28, to Saturday noon, July 3.

This left only three open periods. Wednesday afternoon was employed by many to visit the paint and pigment exposure test panels at Ventnor and Chelsea, just south of Atlantic City, placed there by the Paint Manufacturers' Association and inspected regularly by them in conjunction with Committees E and U of the American Society for Testing Materials. The wood panels have been in place for about two years, the steel panels for less than one year. A few definite results are already evident, but generally the time has been too short to permit of positive comparisons. Thursday evening was made the occasion for a dinner, which was enlivened by numerous witty speeches delivered under the stimulation of toastmaster Willard Smith. A suggestion made at this dinner led to a vote of the meeting, later, in favor of appointing President C. B. Dudley official representative of the Society at the International Congress for Testing Materials to be held at Copenhagen, Denmark, the coming September, and bearer of an invitation to the Union to hold its next congress in the United States. The only other open period, Friday evening, was utilized for an informal engineering smoker, graced by a guaranteed Quakertown colonial antiquity called Fish-House Punch.

The business proceedings of the meeting were small in amount. Messrs. W. R. Webster (Cons. Engr., Philadelphia, Pa.) and W. A. Bostwick (Carnegie Steel Co., Pittsburg, Pa.) were elected to the Executive Committee, which largely directs the destinies of the Society. A good annual report was presented by that committee, showing continued heavy gains in membership (now 1,160) and much work accomplished in formulating specifications and reporting tests of materials and structures. The fly in the ointment is the fact that the finances are running behind, a thing not to be wondered at with the low income of $5 dues per member. The committee suggested two remedies: raising the dues, and soliciting voluntary contributions. The latter expedient prevailed during the Society's early years, and that it has been outgrown was a matter of congratulation. As the membership dues are very low, compared with those of other societies that can hardly claim to be more efficient, it would be natural to raise the dues, but no steps were taken in the matter during the meeting.

Several minor amendments to the by-laws were approved by the meeting and passed to letter-ballot. The same action was taken upon a resolution, introduced by the Executive Committee, expressing the opinion that testing-machines for large structural members are desirable, that Federal aid in their construction is proper, and that such a machine should be made available for the use of private parties.

The technical proceedings were again of amazing volume, as has been the case for some years past. Most of the specification committees presented reports, and as these now have just about exhausted the alphabet with their designation-letters, it is obvious that this part of the proceedings alone was of respectable amount. In addition, some 30 or more technical papers were read, of all grades of complexity and thoroughness and on a remarkably varied range of subjects. Since but few of these were available

to the members in printed form or otherwise before the meeting, their discussion was quite impromptu, and rather less cogent and complete than is desirable.

It appears, moreover, that the study and preparation of specifications, the primary object of the Society, is becoming steadily more a matter of committee work pure and simple. While it is so inherently, yet in the past the specifications proposed have nearly always received active discussion and often fatal criticism at the meetings of the Society. It was markedly less so this year, as illustrated by these striking instances: Committee A, on Iron and Steel, presented a whole batch of new or revised steel specifications, including a never-failing bone of contention, rail specifications; but these were passed by with practically no discussion whatever. Reports of Committee C, on Cement, have always heretofore occasioned lively discussion; this year several amendments submitted were passed to letter-ballot without comment. Finally, the long-expected report of the Joint Committee on Concrete and Reinforced Concrete was submitted, and though there are strong opinions on this report in the profession it was received without a word for or against, and without either query or explanation as to future plans or pending work of the committee. It would appear that the meetings of the Society are reverting entirely to the consideration of technical papers, and that all battling over specifications will in future be circumscribed by the walls of committee-rooms. This is fortunate or unfortunate according to the point of view.

A review, however brief, of the technical proceedings of this meeting must be headed by a remark of praise for the masterly presidential address of Mr. C. B. Dudley (Chemist, Penn. R. R., Altoona, Pa.), entitled "Engineering Responsibility." It proved a worthy successor of the remarkable series of addresses which he has delivered during the past years of the Society's existence, and with them will form a fundamental contribution to the literature of modern engineering practice. An abstract of the address was presented in our last issue, under a slightly different title.

Proceeding now to a review of the other technical proceedings, i. e., the consideration of committee reports and technical papers, space requirements restrict us to very condensed notice indeed, and many papers and reports that contain absorbing interest for large branches of work can be mentioned by title only. However, if we cannot enter into these proceedings very fully, it is to be remarked that those present at the meeting were similarly limited, through sheer overweight of matter offered them for intellectual consumption.

COMMITTEE REPORTS ON SPECIFICATIONS.

A statement of the meeting's work on specifications necessarily can be little more than a brief listing of subjects and the action taken. Though this may be an uninviting record, yet it deals with the most important work of the Society, in fact its primary work, to which the reading and discussion of papers is only auxiliary.

Hard-Drawn Copper Wire.

Committee W, headed by Mr. J. A. Capp (Genl. Elec. Co., Schenectady, N. Y.), brought in a well drawn specification for hard-drawn copper wire, including round wire, grooved trolley wire, and cable made of hard-drawn wire. Its distinctive feature is the omission of twist or wrap tests and of all requirements as to elastic limit; the only mechanical requirements are for tensile strength (49,000 to 67,000 lbs. per sq. in., depending on diameter), and for elongation (0.7% to 2.7% in 10 ins.). Conductivity (900.8 to 910.2 lbs. per mile-ohm) and accuracy of size are the only other requirements. Cable must show 90% of the sum of the strengths of its wires.

Passed to letter-ballot by vote of the meeting.

Fireproof Floor and Partition Tests.

Committee P. on Fireproofing Materials (ch. Prof. I. H. Woolson, Columbia Univ., New York), submitted a tentative specification for fire tests of partitions and a revised specification for fire tests of floor constructions at the meeting of June, 1908. No changes have since been found advisable, and the committee therefore definitely recommends these specifications for adoption as standard.

Passed to letter-ballot by vote of the meeting.

The committee reported also that it is taking up the subject of fireproof column coverings.

Common Building Brick.

Committee D is charged with drawing specifications for building and paving brick. It began its work with building brick. The chairman, Mr. L. W. Page (office of Public Roads, Washington, D. C.), reported that it seemed impossible to specify broadly enough to cover all kinds of building brick, but that a tentative specification for common brick had been prepared. Some of its chief requirements may be stated as follows (from memory):

Modulus of rupture to be measured on 7-in. span, brick lying flatwise, center load, the modulus being calculated

$$f = \frac{3Wl}{2bd^2}$$

Compressive strength to be determined by crushing flatwise one of the halves obtained in the transverse test; the line of break in the transverse test must be within 1 in. of the middle of the brick or the test repeated.

Brick to be tested for both transverse strength and crushing strength (1) when dry; (2) saturated; (3) after freezing and thawing 20 times. The following figures fix minimum and average values to be met by the brick:

	Modulus of Rupture, lbs. per sq. in.		Crushing Str., lbs. per sq. in.	
	Min.	Av.	Min.	Av.
Dry	325	400	2,500	3,000
Saturated	225	275	2,000	2,500
After frzg. & thaw'g test	225	275	2,000	2,500

The 24-hr. absorption should not exceed 15%.

Two vital objections were offered to this specification. Mr. J. E. Howard (Watertown Arsenal, Watertown, Mass.) argued that the requirements for crushing strength are needlessly low, which would tend to cast discredit on brick; and that absorption should be obtained by the accurate method of finding the specific gravity of brick dust and computing the volumetric porosity therefrom, inasmuch as the amount of water absorbed is a variable quantity, increased with time, etc., and cannot be an accurate measure of porosity. Mr. R. L. Humphrey (U. S. Geol. Survey, St. Louis, Mo.) also objected to the unsatisfactory way of specifying absorption, and further objected to the flatwise compression test as meaningless; indeed he objected to any requirement of compression of brick. The objections were answered by Prof. A. N. Talbot (Illinois Univ., Urbana, Ill.) and Mr. Page, and the specification was accepted by the meeting for further action.

Cement.

Two small amendments to the present standard cement specification of the Society were reported by Prof. George F. Swain (Mass. Inst. of Tech., Boston, Mass.), the chairman of Committee C. The requirement for specific gravity of Portland cement, 3.10, is to be changed to permit expressly that cement falling below this value may be retested after ignition at low red heat. The tensile strength requirements both for natural and for Portland cement, which last year were made indeterminate by fixing a range, the middle point of this range being the recommended minimum, are now clarified by fixing single minimum values equal to the middle points of the ranges fixed last year.

These amendments were accepted by the meeting for a letter-ballot vote of the Society.

STANDARD SAND.—Mr. R. L. Humphrey, secretary of Committee C, reported that arrangements have been made for a supply of standard Ottawa sand for tests of cement. The Ottawa Silica Co., Ottawa, Ill., will furnish this sand in uniform packages, under the supervision of the committee, and it is expected that the price can be made so low as to render this sand available everywhere for testing.

Test Methods for Bituminous Materials.

A specification for test methods for bituminous materials has been prepared by Committee H, and was reported by the chairman, Mr. P. Hubbard (Office of Public Roads, Washington, D. C.).

Passed to letter-ballot by vote of the meeting.

Methods of Testing.

Prof. Gaetano Lanza (Mass. Inst. of Technol., Boston, Mass.), chairman of Committee K, on Standard Methods of Testing, presented an extended report setting forth the proper methods of making mechanical tests. The report is in some respects a provisional one, and was therefore laid on the table for one year, to give time for criticism or improvement if found desirable.

Pig Iron.

Committee B, on Specifications for Cast-Iron and Finished Castings, reported only on a specification, or rather a code, for ordering and sampling pig-iron. The code for ordering is as follows: The specification for each element comprises a capital letter giving the name of the element, and a lower-case suffix denoting the quantity specified. These are summarized in the following table:

	Code Letter.	Quantity and Percents denoted a	e	i	o	u	y	h
Silicon .... (0.25 allowed either way)	L	1.00	1.50	2.00	2.50	3.00	3.50	....
Sulphur ... (maximum)	S	0.04	0.05	0.06	0.07	0.08	0.09	0.10
Total carb.. (minimum)	C	3.00	3.20	3.40	3.60	3.80	....	....
Manganese. (0.20 allowed either way)	M	0.20	0.40	0.60	0.80	1.00	1.25	1.50
Phosph.... (0.15 allowed either way)	P	0.20	0.40	0.60	0.80	1.00	1.25	1.50

NOTE: A further suffix x denotes percentage halfway between quantity letter and next higher value, i. e., Pex denotes Phosphorus 0.50% desired, permissible range 0.35 to 0.65%.

NOTE: In case of Phosphorus and Manganese the percentage stated may be used as maximum or minimum figures if so stated expressly.

EXAMPLE OF USE OF CODE: Li Se Ca Mo Pi specifies pig with Silicon 2.00, Sulphur 0.05, Total Carbon 3.00, Manganese 0.80, Phosphorus 0.60, with variations allowed as above noted.

The specification also recommends that market quotations be based on iron with 2.00% silicon (0.25 either way) and 0.05% sulphur (maximum). For acceptance tests, one sample shall be taken from every 4 tons, drillings from all samples mixed for the whole car, and the standard methods of the American Foundrymen's Association used for analysis.

This specification has been agreed upon between Committee B and the American Foundrymen's Association.

As formal signatures have not yet been attached by all members of the committee, the meeting voted to pass the specification to letter-ballot if all members of the committee accept it, in order that the code may be offered to the International Congress in September.

Steel Plates, Rails, Etc.

Committee A, on Specifications for Iron and Steel, ch. Wm. R. Webster (Cons. Engr., Philadelphia, Pa.), submitted revised specifications for the following materials:

Structural steel for ships.
Structural steel for buildings.
Boiler-plate and rivet steel.
Steel splice-bars.
Steel tires.
Bessemer steel rails.
Open-hearth steel rails.

Mr. E. D. Meier, of the committee on Boilers made a protest against allowing 0.05% sulphur in flange steel (general boiler steel), and claimed that 0.04% was a feasible limitation. The opinion of the meeting did not support this view, however, nor did it support Mr. H. V. Wille's plea for .035% limit on sulphur for firebox steel and provision for tensile tests transverse to the direction of rolling.

The whole group of specifications was passed to letter-ballot, by vote of the meeting.

Linseed Oil.

A provisional specification for linseed oil was brought in at a late hour by a subcommittee of Committee E, on Preservative Coatings for Iron and Steel. It is based on check analyses of specially obtained samples of oil, made by many chemists. Final results will be available by next year, and a specification will probably be recommended for adoption at that time. It was remarked by the chairman of the subcommittee, Mr. G. Thompson (National Lead Co., New York City), that drying coefficient and oxygen absorp-

tion tests are very unsatisfactory at present, and that suggestions for improvement in these points are desired.

OTHER COMMITTEE MATTERS.

OBTAINING STANDARD SAMPLES OF IRON ORES.—The Bureau of Standards is about to take up with various interested societies and organizations the subject of preparing and analyzing samples of various iron ores (presumably for use as check samples in ore analyses). For this purpose it desired to engage the aid of the society in procuring and selecting samples of iron ores. The matter being brought before the meeting, it was voted to refer to the Executive Committee the question of taking part in the work.

It is to be remembered that the Bureau of Standards now prepares and distributes standard samples of cast-iron, carefully analyzed and therefore of known, uniform composition, as check samples for use of analysts just as now proposed for iron ore. Mr. Thos. D. West originated this undertaking for cast-iron, transferring it to the Bureau a few years ago; these standard samples have proved very useful in furthering uniformity of chemical analysis of iron.

STUDY OF EXPLOSIVES. Prof C. E. Munroe (George Washington Univ., Washington, D. C.) suggested to the Society in a well-argued address the need for a study of explosives with a view toward unifying practice, reducing present unnecessary risks, and fixing standards of quality. Col. B. W. Dunn, of the American Railway Association's Bureau for the Safe Transportation of Explosives, etc., seconded this appeal. The meeting thereupon voted to refer the matter to the Executive Committee for consideration and action.

SPECIFICATIONS FOR WROUGHT-IRON.—Hitherto both wrought-iron and steel have been in charge of Committee A, which has already prepared a considerable number of specifications for the Society. In this year's report of that committee, however, it reported that no unanimity of opinion could be reached in the matter of specifications for wrought-iron, because the many different uses of the material seemed to demand different standards of quality. The committee recommended that it be relieved of further consideration of wrought-iron, and a separate committee be established to deal with this subject, Committee A to be renamed Committee on Steel.

This was referred to the Executive Committee for action.

SPECIFICATIONS FOR COAL.—According to a progress report of Committee O, by its chairman, Mr. J. A. Holmes, the committee has organized in a number of distinct sections, each to deal with coal for a single kind of use, as steam boiler use, gas-producer use, etc. The report also outlined a number of the considerations which will guide the subsequent work, all occupying a high plane and promising very thorough and progressive work.

COLD-DRAWN STEEL.—Mr. C. E. Skinner (Westinghouse Elec. & Mfg. Co., Pittsburg, Pa.) reported that his committee on specifications for cold-drawn steel had divided its work to cover separately shafting, axle stock, automatic screw stock, etc., and has eliminated wire from present consideration. To obtain a definite basis for further action it will first make a series of tests on automatic screw stock, and in this connection it desires suggestions for a simple test for ease of machining.

WATERPROOFING CONCRETE.—Committee S, on Waterproofing, made an interesting progress report. It has two subcommittees on waterproofing concrete, the first dealing with waterproofing by the incorporation of non-bituminous substances, the second dealing with bituminous surface treatments or fabrics.

Subcommittee A reports the

positive information that with even such ungraded materials as crushed quartz or standard Ottawa sand, no difficulty is experienced, in a laboratory with its care in operating, in obtaining waterproof mortars in such lean proportions as 1 cement to 4 sand. There is no excuse for failure when a fairly graded natural sand is used with even leaner mortars, confirming the fact that the reason ordinary field concrete mixtures need waterproofing treatment is due either to the use of poor materials or poor proportioning or bad handling, or cumulatively to all of these combined.

We think it has been demonstrated and will be generally admitted that with proper materials, proper proportioning and proper handling of the subsequent mixtures, these need no additions of foreign substances to become initially waterproof; that when such addition is needed for reasons above stated, the desired end can be and is most easily secured mechanically through proper void-filling, compensating for the poor proportioning or the poor quality of the original constituents. Any chemical action claimed for mixtures operating towards securing waterproof concrete is apparently nil. Since void-filling is to be sought as the panacea of waterproofing ordinary field concrete, comparative tests through the addition of percentages of colloidal clay or hydrated lime and the various advertised waterproofing compounds have been carried on continuously, with the conclusion at this date that no general results are obtainable from the additions of patented or proprietary compounds which cannot be obtained equally well through the use of colloidal clay or hydrated lime. Additionally, it seems to be very generally proved by corroborative tests, results of which will be submitted later when long-time results will be available, that, as stated in last year's report to be likely, a weakening in tensile strength in time followed the use of many of the patented compounds, which effect is not generally marked when colloidal clay or hydrated lime is used.

The other subcommittee has instituted tests as follows: Tanks 18 × 12 × 9 ins. were built of 1:3:5 concrete, and manufacturers of waterproofing finishes were asked to protect the inside with their materials. Then the inside was surfaced with very lean cement mortar. The 3-in. bottom had previously been punched with 5-16-in. holes, to give earlier indications. Then the tanks were filled with four liquids: Astoria gas drip; New York subway seepage water; the same with 5% kerosene; city water with 23% potash. The results of these tests are not yet complete, as the attacking effect was not positive in all tanks, but will be reported later.

Mr. W. A. Aiken (H. S. Spackman Engrg. Co., Philadelphia, Pa.) is chairman and Mr. A. W. Dow (Cons. Engr., New York City) is secretary of the committee.

CONCRETE AND REINFORCED CONCRETE.—The Joint Committee on Concrete and Reinforced Concrete, which reported last January to the American Society of Civil Engineers through that society's representatives on the Joint Committee, reported to the American Society for Testing Materials at the present meeting. It presented the same report which went before the Civil Engineers as a majority report, but the minority report made to that society was not submitted here.

It is called a "Progress Report," and is characterized thus:

The report herein presented embodies the present judgment of the Committee concerning the proper use of concrete and reinforced concrete.

Formulas, rules and stress values for designing are contained in it, together with numerous general remarks. No experimental data are presented.

The committee made no statement as to what further work is contemplated, or as to what kind of Final Report will be prepared, if any.

On motion of Mr. R. L. Humphrey (secretary of the Joint Committee), the report was accepted by the Society for publication in its Proceedings for the information of its members. It is now available in pamphlet form.

PRESERVATIVE COATINGS FOR IRON AND STEEL.—Committee E has had a subcommittee working on the subject of linseed oil, and very elaborate experimenting has been done by it to determine what tests to specify and what values to fix as standard for pure linseed oil in the different tests. A detailed report of this work, so far as analyses had been received, was submitted in printed form. Later in the meeting the subcommittee presented a provisional specification, for further elaboration and proof during the coming year, as noted above under the head of Specifications.

HAVRE DE GRACE BRIDGE PAINT TESTS.—The main part of Committee E has been concerned in periodical inspections of the large-scale exposure test of different bridge paints on the Havre de Grace (Md.) bridge of the Pennsylvania R. R. The committee reports that in nearly every case the paint on both the bridge and the check plates is giving excellent protection, after now 2½ years' exposure. Some differentiation can already be made, and most of the paint films are showing the effects of time and exposure. It is expected that in another year some definite conclusions can be drawn.

WOOD AND STEEL PANEL TESTS AT ATLANTIC CITY.—The exposure tests of wood panels at Atlantic City, conducted by the Paint Manufacturers' Association, are inspected jointly with Committee E. Some results are already apparent there, for example, that pure lithopone and paints containing as low as 60% lithopone have failed entirely. Pure white-lead has chalked more than some and less than others of the composite whites.

Steel panel tests were started in conjunction with Committee U (on Corrosion), also at Atlantic City, within ¼ mile of the beach. These panels are painted with linseed-oil paints carrying some two or three dozen different pigments, and there are corresponding panels of Bessemer and open-hearth steel and pure charcoal iron. The pigments were selected with a view to getting information on the influence of the pigment on corrosion of the metal, and in fact this is the entire object of this test. No marked differences are yet observable between corresponding panels of different metals, but some of the paints have deteriorated very much more than others. These panels have now been exposed for nearly a year.

In discussion the objection was made that these steel-panel tests will be only misleading as to the value of pigments, since if a given pigment is unsatisfactory in the proportions, etc., as used in the test, it may give quite the contrary result with even a slight admixture of other substances. This was acknowledged, and the statement made that the sole purpose of these tests is to show whether pigments in paints under actual exposure will give the same relative results as they do in a laboratory test devised by Mr. G. W. Thompson for discriminating between different pigments as to their effect in stimulating or retarding corrosion. It was stated by Mr. Thompson, of the committee, that if the panel tests were interpreted to condemn any pigment they are unwise. Another speaker reported that certain plate tests, now 2 years old, showed best protection by a paint containing a mixture of two pigments generally classified as stimulators.

RUSTING.—Committee U on Corrosion has started exposure tests of fence wire at Pittsburg, begun last fall, and jointly with Committee E (Preservative Coatings) has taken part in the exposure tests of steel and iron panels noted above.

This report led to very diversified discussion on the relative corrosion of puddled wrought-iron and soft steel. The usual hopeless conflict of testimony and opinion was varied by a striking instance brought forward by Mr. J. P. Snow. A signal bridge made part of wrought-iron and part of steel showed astonishing differences of amount of rusting, the wrought-iron being many times the better. The only weakness of the case, as objected by Prof. Walker, is that the two metals having different electrochemical potentials, one would tend to induce corrosion of the other, whereas if they had been entirely separate they might have given about equal performance.

METHODS OF ANNEALING STEEL.—Prof. H. M. Howe for Committee F, on Heat Treatment of Iron and Steel, presented a very interesting progress report which gives methods of annealing ordinary carbon steel containing not over 0.90% carbon.

1. Carbon Steel Castings Containing 0.50% Carbon or Less.—Heat to 1,000° C. (1,832° F.) for twelve hours and cool slowly. The temperature should not at any time rise above 1,050° C. (1,922° F.).

2. Rolled and Hammered Steel.—To relieve stress, the effects of cold or cool working, etc., heat to slightly above the absorption point (Ac1, about 730° C. [1,346° F.]), and cool slowly.

The absorption point may be recognized easily, because as the temperature rises past it the rate of rise is momentarily lessened, i. e., the rise itself is slightly retarded spontaneously, and may even be wholly arrested. If the reduction in rolling or hammering has been great enough, so that the coarsening of the grain at the high temperature to which the steel had been heated prior to such rolling or hammering has been well effaced, then carrying the heating for this annealing operation far above the absorption point not only is not beneficial, but is even slightly harmful, and the harm increases with the distance above the absorption point to which the temperature is raised, and also with the percentage of carbon. In other words, heating needlessly far above the absorption point is much more hurtful to high-carbon than to low-carbon steel.

3. To Remedy the Effects of Overheating, as for instance when the metal has for any reason been allowed to cool from a temperature far above the recalescence point, Ar1., without simultaneously undergoing substantial reduction of area by rolling, hammering, or equivalent treatment preventive of harm. In order to remove the effects of such undisturbed cooling from an unduly high temperature, the steel should be reheated to slightly above Ac3, and cooled slowly. To raise the temperature far above Ac3 tends to undo the good done by heating to just above Ac3. If the steel has been overheated very highly or for a very long time, then the length of time during which it must be held above Ac3 should be increased proportionally. This is better than to carry the reheating to a still higher temperature.

4. How to Recognize the Temperature Ac3.—When the carbon content is between 0.45 and 0.90%, Ac3 is the temperature at which the steel loses its magnetism. In steels with carbon content less than 0.45%, Ac3 is above the temperature of the loss of the magnetism. For those steels Ac3 can be calculated roughly by the formula:

Ac3 = 900° − C × 200° in Centigrade degrees.
= 1,652° − C × 360° in Fahrenheit degrees.

In this formula C is the percentage of carbon in the steel. For instance, if the carbon content is 0.30%, then Ac3 is about 900° − 200° × 0.30 = 840° C., or 1,652° − 360° × 0.30 = 1,544° F. This formula indeed applies also to steel with between 0.45 and 0.90% of carbon.

5. Condensed List of the Critical Temperatures Referred to.

The Recalescence, or Ar1, about 700° C. (1,292° F.) independently of the carbon content.

The Absorption Point, or Ac1, about 730° C. (1,346° F.) independently of the carbon content.

Ac3. For steel containing less than 0.90% of carbon, Ac3 can be calculated roughly by the formula:

Ac3 = 900° − C × 200° in Centigrade degrees
= 1,652° − C × 360° in Fahrenheit degrees

in which C = the percentage of carbon.

In case of steels containing between 0.45 and 0.90% of carbon, Ac3 corresponds to the loss of magnetism on heating.

CLAY AND CEMENT SEWER PIPE.—A progress report was submitted by Mr. Rudolph Hering (Cons. Engr., New York City), chairman, outlining a program for further work, and sketching the points which require consideration in specifying for sewer pipe. The desire was expressed by several, in discussion, that the committee should also consider the subject of drain tile, because specifications for drain tile are often asked for and a sewer-pipe specification would hardly be accepted for the purpose. But the matter was left to the judgment of the committee.

Several committees did not present reports. These included: R, on Uniform Specifications for Boilers, ch. E. D. Meier (Heine Safety Boiler Co., New York City); M, on Specifications for Staybolt Iron, ch. H. V. Wille (Baldwin Loco. Wks., Philadelphia, Pa.); and Q, on Specifications for the Grading of Structural Timber, ch. H. von Schrenk (Cons. Engr., St. Louis, Mo.).

Technical Papers.

The most interesting of the papers read at the meeting were those which reported test results. Some of these brought definite new information, while others fell short of final results, but indicated progress.

CASTING IN IRON MOLDS.—To begin with the paper that had perhaps least direct relation to the society's tendencies, Mr. E. D. Custer brought forward a distinct novelty. In a paper, "The Permanent Mold and Its Effect on Cast Iron," he told of his experiences in making castings in iron molds for repetitive use. Numerous remarkable observations on the behavior of iron cast under these conditions were cited. The immediate practical fact is that excellent castings are produced without difficulty, using any grade of iron that is suitable for ordinary foundry purposes, if only the precaution is observed to remove the casting from the mold as soon as its outside has solidified, while it is still glowing hot. No chill is then produced, as the casting cools slowly through the recalescence stage, and a distinct gain is had in the attainment of great homogeneity, because the metal is solidified by the mold quickly enough to prevent segregation. Wonderful speed in casting is recorded; the specimen figures given by the author were, a soil-pipe every 15 sec., and a brakeshoe every 20 sec., in the same mold, the cores also being of iron.

RAILWAY IMPACT TESTS.—An unusual subject was discussed by Col. B. W. Dunn, under the title "Measurement of Impact Stresses." He described experiments made by the Bureau for the Safe Transportation of Explosives (Am. Ry. Assn.) to determine the impact pressure generated by tightly blocked lading in a freight-car during light collisions such as occur in coupling cars. (1) By fixing a longitudinal pipe full of water in the car and reading pressures on a gage attached to its forward end; (2) by arranging a weight to slide longitudinally, held against forward movement by the spring of a gage; readings of impact forces per pound weight were obtained. Those measured by the first method were always the higher, and when coupling to a braked cut of cars at even the low speed of 4 min. per hr. the pressure was as high as 7 to 9 lbs. per pound of lading. The bearing of this on the transportation of explosives is as follows: When boxes of dynamite or kegs of powder are packed loose in the car, they are thrown around and damaged during the transport, so that it has been necessary to prescribe that the load must be blocked tight against braces. But then the inertia of the whole lading comes against the forward row of packages (next the braces), and from the above-cited tests it appears that if the load in half of the car is 15,000 lbs., with say 12 packages resting against the braces, the inertia effect of 8 lbs. per lb. will (neglecting friction) bring a pressure of 10,000 lbs. on each of the forward packages during a coupling shock. Such pressures are just about enough to crush a box of dynamite, and more than enough to deform a powder keg. It is evident that further evolution in packing explosives in cars is needed.

Incidentally, the author described experiments made to determine the resistance to impact of copper plugs used for measuring explosion pressures in large guns. By delicate impact tests it was determined that such a plug exerts slightly greater resistance to very rapid compression than to static (testing machine) distortion—perhaps 10 per cent. greater resistance when the compression takes place in two or three one-thousandths of a second.

GALVANIZED IRON.—"Testing Galvanized and Other Zinc-Coated Iron" was treated in remarkably instructive fashion by Prof. William H. Walker (Mass. Inst. of Tech., Boston). It seems that immersion in copper-sulphate solution is the only test now available for determining the quality of a zinc coating. The deficiencies of this test, the urgent need of fuller tests, and studies tending toward new and better tests, were sketched by the author. A vital point is this: The quality of the zinc coating is the important thing, while the quality of the iron underneath is probably of no moment. We hope to be able to reproduce the paper fully at another time.

TWO NEW TESTING MACHINES.—A testing machine which obtains its pressure by a hydraulic jack and weighs this pressure in a novel manner was furnished to Rensselaer Polytechnic Institute by T. Olsen & Co. recently. This is of 600 tons capacity. A closely similar machine of 5,000 tons capacity for the Geological Survey is nearly completed (see Eng. News, Feb. 11, 1909). The 600-ton machine was described by Mr. T. Y. Olsen at the meeting. The pressure of the water in the press is weighed by a set of ordinary weighing levers which act on a flexible diaphragm in the cylinder, the area of the diaphragm being just 1-25 that of the cylinder cross-section. The makers claim for the weighing system an accuracy exceeding 1-3 of 1 per cent. The machine was calibrated by weighted levers for low loads, and on extending this upward and checking against the Illinois 300-ton lever machine a very exact coincidence was obtained. The machine is so sensitive as to respond to the pressure of the hand when balanced under load.

Mr. Olsen read a second paper on "A Machine of New Design for Hardness Test." This machine has compound levers 1:100 to which a constant weight is applied after the specimen is in the machine. These levers press a 3-8-in. steel ball upon the specimen tested, and at the same time a multiplying pointer indicates the indentation produced by the ball.

BITUMEN TESTING INSTRUMENTS.—Mr. H. Abraham (Standard Paint Co., Bound Brook, N. J.), described two carefully designed instruments for tests on bituminous materials. One of them, a needle instrument for testing hardness or penetrability, represents a distinctly new idea in this direction, as the needles used, which have a mushroom-shaped flat head, are pressed into the material at a definite rate of penetration (1 cm. in 1 min.), to a definite depth (1 cm.), and the maximum pressure required for this is recorded automatically. The square root of this pressure is taken to designate the hardness numerically. The number 100 represents a load of 10,000 grams on the smallest needle used, 0.01 sq. cm. in area, and as this represents a pressure of over 14,000 lbs. per sq. in., rather hard substances can be represented on this scale of hardness. This instrument has been called the Consistometer.

Other instruments for testing oils or bitumens were described by Mr. A. W. Dow (New York City), "A Viscometer for Heavy Road Oils"; by Mr. F. P. Smith (New York City), "A Machine for Testing the Ductility of Bituminous Paving Cements"; and by Mr. C. N. Forrest (Maurer, N. J.), "A Further Development of the Penetrometer as Used in the Determination of the Consistency of Semi-Solid Bitumens." These papers, unfortunately, were read by title only, but will be printed in the Society's Transactions.

Several other papers on oils and bitumens were presented, most of them being not read. These were:

"The Effect of Free Carbon in Tars from the Standpoint of Road Treatment," by Mr. P. Hubbard (Office of Public Roads, Washington, D. C.).

"Methods for the Examination of Bituminous Materials in Use in Highway Construction," by Messrs. C. Richardson and C. N. Forrest (Maurer, N. J.).

"Bituminous Materials for Use In and On Road Surfaces, and Means of Determining their Character," by Mr. C. Richardson.

"Notes on Testing Turbine Oil," by Mr. R. Job (Philadelphia, Pa.).

"Conditions Affecting the Determination of Carbenes in Bitumens of Carbon Tetrachloride," by Messrs. C. N. Forrest and D. B. W. Alexander (Maurer, N. J.).

CARBON IN ROAD OILS.—Of these Mr. Hubbard's paper gave a large series of tests of bituminous oils having different percentages of "free carbon," with and without other admixtures, to determine binding power, etc. The general conclusion was that cheapness and efficiency dictate using oils of low amount of free carbon, mixing it with limestone-dust filler if necessary. But results of practice, quoted by Mr. A. H. Blanchard (State Board of Pub. Roads, Providence, R. I.) and others indicated that some oils of high free carbon content give excellent service, as good as other oils of low free carbon.

TURBINE LUBRICATING OIL—Mr. Job recommended that oils for lubricating steam-turbine bearings or otherwise in exacting continuous use (in circulating oil systems) be tested to determine how far it is free from amorphous waste. His tests are: (1) Heat to 450° F. and hold for 5 mins.; the oil should show only slight darkening, while marked darkening indicates unsuitable oil. (2) Shake 1 oz. oil with 2 oz. boiling water in a 4-oz. bottle, for one minute; on standing, the oil should separate rapidly and no waxy substance should appear at the junction of oil and water.

Some discussion of this paper by President Dudley and Mr. P. H. Conradson (Galena Signal Oil Co., Franklin, Pa.) referred the deterioration of oils in long service to absorption of oxygen; the acidity appears to increase during this action.

The subject of oils for highway treatment led to quite a bit of discussion, but perhaps the only vital point of this was emphasis of the present need for studying failures as well as successes in road treatment, to collect data on which more definite knowledge of the subject may be built.

BENDING TESTS OF I-BEAMS.—Prof. Edgar Marburg (Univ. of Penna., Philadelphia) recently conducted an excellent series of bending tests of standard and wide-flange I-beams, the latter being 15-in. to 30-in. beams rolled by the Bethlehem Steel Co. on its Grey universal beam mill.

It developed that corresponding depths of the different kinds of beam showed roughly equal utilization of their respective section moduli, up to 24-in. beams. The 30-in. Bethlehem beams showed distinctly less effective utilization of section modulus, and their computed extreme-fiber stress at elastic limit was well below 15,000 lbs. per sq. in. Even the 15-in. beams, however, developed an elastic-limit stress of only 25,000 average, with many single results but little above 20,000 lbs. per sq. in.

A large number of interesting points was found, which we must pass over for the present. One special feature was that the standard beams (and some of the others) failed by lateral buckling of the top flange, whereas most of the wide-flange beams failed by what the author calls twisting of the web. Another point discovered was the presence of material having very low elastic limit at the junction of web and flange.

It was properly objected, in the discussion on the floor, that I-beams are not meant to be used without lateral support of the top flange on such long spans as were tested; also, that the end support of beams in practice stiffens them against failure by twisting of web. But as both Prof. Marburg and Prof. A. N. Talbot asserted, lateral support is of no influence until the elastic limit is well past. The objection does not seriously impair the great value of the tests, or weaken their showing that I-beams in full-size test have a very low elastic limit.

TESTS OF STEEL COLUMNS.—A curious parallel to these beam tests was afforded by column tests reported by Mr. J. E. Howard under the title "Notes on Tests of Steel Columns in Progress at Watertown Arsenal." This paper, which we expect to abstract later, showed little falling off in strength between length-ratio limits of 30 and 150. The failure values in load per square inch were not very high, however, ranging from 40,000 down to 20,000, while the elastic limits of the columns were of course still lower, averaging below 30,000 lbs. per sq. in. Tubes, rolled H-sections and riveted I-sections were tested, with little difference, and with substantially equal results for flat, pin and round ends.

In discussion Mr. L. S. Moisseiff (New York City) reported that investigation of column strength has recently been made for the Department of Bridges of New York City, though its results have not yet been published. One notable result thereof is the conclusion that for 60,000-lb. steel the proper unit-load for columns is to be based on a value ¾ the allowable tensile stress, with a rather small length-ratio reduction. The formula $p = 12{,}000 - 30\ l/r$ is a fair expression of this conclusion.

(To be concluded next week.)

Smokeless Boiler Plants.

The U. S. Geological Survey has carried on investigations on the burning of bituminous coal without smoke for several years because of the advantages of smokeless combustion in the many public buildings and on naval vessels and also because of the bearing of the problem on the conservation of national fuel resources. In investigating commercial conditions in plants where coal high in volatile matter was burned, a study was made of conditions in 13 of the larger cities in Illinois, Indiana, Kentucky, Maryland, Michigan, Missouri, New York, Ohio and Pennsylvania. Some 500 plants were inspected. This study led to the conclusion that bituminous coals, high in volatile matter, can be burned without objectionable smoke and in plants carrying widely fluctuating loads, where banked fires must be quickly revived and the boilers over them must be forced to their full capacity. Proper equipment, efficient labor and intelligent supervision are given as the essential requirements.

In Bulletin 334, of the Geological Survey, published in 1908, the theoretical points involved were treated by Mr. D. T. Randall, one of the authors of Bulletin 373, just issued. This latter bulletin gives details of the tests on commercial and experimental plants undertaken as a part of the study previously mentioned. The results are summarized by the authors in what follows:

GENERAL CONCLUSIONS ON SMOKE ABATEMENT.—The flame and the distilled gases should not be allowed to come into contact with the boiler surfaces until combustion is complete.

Fire-brick furnaces of sufficient length and a continuous or nearly continuous supply of coal and air to the fire make it possible to burn most coals efficiently and without smoke.

Coals containing a large percentage of tar and heavy hydro-carbons are difficult to burn without smoke and require special furnaces and more than ordinary care in firing.

Briquets are suitable for use under power-plant conditions when burned in a reasonably good furnace at the temperatures at which such furnaces are usually operated. In such furnaces briquets generally give better results than the same coal burned raw.

In ordinary boiler furnaces only coals high in fixed carbon can be burned without smoke, except by expert firemen using more than ordinary care in firing.

Combinations of boiler-room equipment suitable for nearly all power-plant conditions can be selected, and can be operated without objectionable smoke when reasonable care is exercised.

Of the existing plants some can be remodeled to advantage. Others can not, but must continue to burn coals high in fixed carbon or to burn other coals with inefficient results, accompanied by more or less annoyance from smoke. In these cases a new, well-designed plant is the only solution of the difficulty.

Large plants are for obvious reasons usually operated more economically than small ones, and the increasing growth of central plants offers a solution of the problem of procuring heat and power at a reasonable price and without annoyance from smoke.

The increasing use of coke from by-product coke plants in sections where soft coal was previously used, the use of gas for domestic purposes, and the purchase of heat from a central plant in business and residence sections all have their influence in making possible a clean and comfortable city.

RAILWAY ACCIDENT BULLETIN NO. 31, recently issued by the Interstate Commerce Commission, gives the number of persons killed in train accidents during the months of January, February and March, 1909, as 177. The number of persons injured in train accidents was 2,618. These reports deal only with employees on duty and passengers. The number of passengers killed, exclusive of postal clerks, express messengers, etc., was 69, but only 28 of these were killed in train accidents. Of the remaining 41, 28 deaths were occasioned by falling from cars while getting on or off and the remaining 13 were due to other causes not specified. The total number of passengers and employees killed on the railways was 663 as compared with 798 in the last quarter of 1908.

There were 1,042 collisions and 1,242 derailments in the quarter now under review, of which 168 collisions and 145 derailments affected passenger trains. The greatest loss of life in any one accident during the quarter occurred in the collision on the Denver & Rio Grande R. R. Jan. 15, when 20 persons were killed. This accident is ascribed to the misunderstanding of a train order.

A RUNAWAY FREIGHT TRAIN was wrecked June 25 on the Butte County R. R. near Chico, Cal. One member of the train crew was killed and three others were injured. The train was made up of 29 cars, of which 18 were loaded with lumber, and the surviving members of the crew, according to reports, lay the blame for the wreck upon orders given them to run such a long train, the customary limit on this road being 15 cars. The train got beyond control on a steep downgrade and attained a speed so high that it left the rails after traversing a distance of about 1½ miles. The lumber-laden cars were piled together in the ditch, and were consumed by the fire which started from the locomotive and was fed by oil from a tank car. The caboose, in which were five passengers, was saved by the conductor, who, as soon as he realized that the train was beyond control of the locomotive, uncoupled the caboose and stopped it with its own brakes.

A COAL MINE EXPLOSION at Tollerville, Colo., July 6, killed nine men in the mine of the Cedar Hill Coal & Coke Co. The explosion occurred while the men were descending the shaft in the cage. The shaft was partially wrecked and some of the men were killed by the force of the explosion while the others were suffocated by gas.

A FOUNDATION TRENCH CAVED IN at the Alexandra dock at Newport, Monmouthshire, England, July 2. There were 50 workmen in the trench at the time and about 30 were buried by the cave-in, but three of those buried were taken out alive and four others are reported to be alive, being partially protected by timbers from the earth pressure, but are beyond the immediate reach of the rescuers. The trench was dug for a concrete foundation to be used in connection with the projected enlargement of the dock. It was 300 ft. long, 30 ft. wide and 60 ft. deep, and, at the time of the accident, the excavation had been completed and preparations were in progress for laying the foundation. The cave-in is ascribed to the collapse of shoring timbers, although reports state that they had been inspected daily throughout the work.

AN EXPLOSION OF BLASTING POWDER at Toplits, Utah, July 3, killed three miners and seriously injured six others. Reports state that the explosion involved 750 kegs of black powder.

THE HEADINGS OF THE GUNNISON TUNNEL, on the Uncompahgre Valley irrigation project, U. S. Reclamation Service, met on July 6. Work has been going on there for the past four years and although record progress has been made at times, there has been a great deal of trouble with running water and bad ground. The tunnel is some six miles long with a finished concrete-lined section of 10½ × 11½ ft. It is to carry the waters of the Gunnison River through a mountain range into the Uncompahgre Valley, where they will be used for irrigation.

FOUR SERIOUS CANAL BREAKDOWNS have been reported within the past two weeks, two on the Erie Canal, one on the Hennepin Canal and one on the Illinois & Michigan Canal. At Culvert 49 on the Erie Canal, near Spencerport, N. Y., there have been several leaks culminating on June 26 in a break in the bank of the canal at that point of such an extent that it became necessary to draw off the water from the level, a particularly long one of 62 miles from Brighton to Lockport. A large number of boats were stranded and through traffic suspended for a few days. The trouble occurred at a masonry culvert that had been widened with a concrete addition in order to take care of the Barge Canal enlargement. Charges have been made that the leaks and consequent breaks are due to poor construction and backfilling to this culvert and an investigation is now in progress. Another break on the Erie Canal occurred on June 29 near Fairport, N. Y., and traffic was stopped for a day or two pending repairs.

The break on the Hennepin Canal came on June 28, near the aqueduct where the canal passes over Green River in Illinois. At this place the canal level is some 7 ft. above the surrounding ground. According to newspaper reports the damage is so serious that the entire length of the canal, which runs from Hennepin, Ill., to the Mississippi below Rock Island, is out of commission and it is expected that two months will be needed for repairs. A break of lesser importance occurred at this same location six weeks previous and had just been repaired.

The newspaper accounts of the break in the Illinois & Michigan Canal are as follows:

On June 29 a cloudburst took out a stretch of the canal bank 60 ft. long at a point about six miles below Morris, Ill., and emptied the entire section between Channahon and Marseilles. The section washed out was about 8 ft. high and 60 ft. in length. The canal formed a mighty torrent that poured first into a big quarry hole and then backed up and flooded all the surrounding country. Fields were under water through all the section between Morris and Seneca. One small creek which had never been known as a navigable stream rose 8 ft. in ten minutes. There were no boats in that section of the canal when the cloudburst came. A few rowboats were tied up at different points, but no damage was done. In the section between Channahon and Joliet there were a number of boats, including two on their way to the gulf. These are compelled to await the repairs on the channel.

DRAWBRIDGES SWUNG BY THE WIND form a curious kind of accident which has occurred in two instances on the Chicago Drainage Canal. A few months ago the Ninth St bridge at Lockport, which (like all the bridges) has as yet no turning machinery, was being swung by hand when it got beyond control and was blown round by the wind. This bridge is about 310 ft. long, with 195 ft. 10 ins. for the longer arm. A similar case occurred some years ago at the 16th St. bridge at Lockport. In neither case was any damage done.

A SECTION HAND WAS ELECTROCUTED on the line of the Erie R. R. at Hamburg, N. Y., near Buffalo, on June 29. The man had been measuring the height of telegraph wires with a linen tape which had been thrown over the wires. At a trolley bridge at Hamburg several transmission cables were to be measured and as the tape was thrown over them the section hand, holding one end, was killed. It is stated that the tape used was probably reinforced with eight or ten brass wires to prevent stretching and that the section hand was ignorant of the danger of his action.

FINES FOR POOLING BIDS have been levied against four bridge companies in connection with two bridges built by the city of Boston, Mass. The companies pleaded guilty to the indictments. In the Brookline St. bridge

the Boston Bridge Co. and the Groton Bridge Co. were each fined $750, and the Canton Bridge Co. and the Owego Bridge Co., $1,000, respectively. For collusive bidding on the Atlantic Ave. drawbridge the first two of the above companies were each fined $750 and the third again $1,000, but the Owego Bridge Co. was not involved in that case. It is stated that there are still a number of indictments out against individual steel men and companies in connection with the cases.

LAND RECLAMATION BY HYDRAULIC FILLING is to be employed by the Board of Park Commissioners at Omaha, Neb., and a suction dredge has been ordered for filling the low lands adjacent to Levi Carter Park. It is estimated that it will take six years to complete the work now contemplated. The dredge is designed to make a cut 100 ft. wide and 24 ft. deep, and its capacity is 250 cu. yds. per hour. A 12-in. dredging pump is used, with 2,000 ft. of 12-in. discharge pipe line. About 800 ft. of this pipe will be carried by pontoons, and the sections will be connected by rubber hose 12 ins. diameter and 6 ft. long, specially arranged to withstand the abrasive effects of the dredged material.

THE NIAGARA FALLS LIGHTING CO. has been denied permission, by the Public Service Commission of the Second District, State of New York, to exercise the rights granted by the city of Niagara Falls, to issue $100,000 stock and $300,000 bonds and to compete in the electric-current supply business in Niagara Falls. The necessary permission was refused as the Commission held that the existing system was adequate and efficient. The present local company, it is stated, had been required to reduce the price of arc-lamp operation from $65 to $52 per year.

Personals.

Dr. Charles W. Eliot has been made President-Emeritus of Harvard University.

Mr. Clifford M. Stegner has been appointed Structural Engineer in the Department of Buildings of Cincinnati, Ohio.

Mr. Hugh D. Lumsden, M. Can. Soc. C. E., Chief Engineer of the National Trans-Continental Ry. of Canada, has resigned.

Mr. Charles E. Doyle, General Manager of the Chesapeake & Ohio Ry., has been made Vice-President and General Manager.

Mr. W. H. Petersen has been appointed Principal Assistant Engineer of the Chicago, Rock Island & Pacific Ry., with headquarters at Chicago, Ill.

Mr. A. C. Ridgway, General Manager, and Mr. W. D. Lee, Superintendent of the fourth division, both of the Denver & Rio Grande R. R., have retired.

Mr. A. J. Poole, formerly General Master Mechanic of the Seaboard Air Line Ry., has been made Superintendent of Motive Power. His former position has been abolished.

Mr. George R. Huntington, General Superintendent of the Minneapolis, St. Paul & Sault Ste. Marie Ry., has become General Manager, succeeding Mr. Edmund Pennington, who is now President.

Mr. W. Bollons, formerly Division Engineer of Maintenance of Way of the Oregon R. R. & Navigation Co., has become Division Superintendent at Portland, Ore., succeeding Mr. D. W. Campbell.

Mr. Gerard Van Tassel, Superintendent of the Harlem division of the New York Central & Hudson River R. R., has been made also Superintendent of the Putnam division, extending from Brewster, N. Y., to 155th St., New York City.

Mr. F. W. Hanna, Engineer in charge of the Technical Section of the U. S. Reclamation Service at Washington, D. C., for the past four years, took charge on June 1 of the Payette-Boise Project as Project Engineer, with headquarters at Boise, Idaho.

Mr. Charles E. Burgoon, formerly Assistant Inspector of Electric Light Plants, U. S. Treasury Department, has been transferred to the position of Assistant to Mr. W. A. E. Doying, Inspecting Engineer, Isthmian Canal Commission, Washington, D. C.

Mr. G. D. Brooke, Assoc. M. Am. Soc. C. E., formerly Assistant Division Engineer of the Baltimore & Ohio R. R. Co. at Pittsburg, Pa., has been appointed Division Engineer of Maintenance of Way at Baltimore, Md., to succeed Mr. Wm. Trapnell.

Hon. Ray S. Reid, State Waterways Commissioner of Wisconsin, whose report was printed in our issue of Jan. 28, is to go to Europe to plan the route for the National Waterways Commission, which is to spend the summer studying European waterways.

Mr. C. W. Sturtevant, M. Am. Soc. C. E., formerly Engineer with the Atlantic, Gulf & Pacific Co., has become associated with the Stewart-Kerbaugh-Shanley Co., of New York City, and will take charge of their contract No. 12 on the New York State Barge Canal at Brewerton, N. Y.

Mr. Henry R. Buck, M. Am. Soc. C. E., and Mr. Paul Sheldon have associated themselves under the firm name of Buck & Sheldon, Incorporated, for the purpose of carrying on and enlarging the engineering practice of Mr. Buck. Their offices are in rooms 724-7, Connecticut Mutual Bldg., Hartford, Conn.

Mr. Robt. E. McDonnell, of Kansas City, Mo., has started upon a four months' trip to Germany, France and England, where he will investigate water works and sewage disposal plants with a view toward the preparation of a report relative to the contemplated improvements in the water system in Kansas City.

A State Waterways Commission for Missouri has been appointed by Gov. Hadley in accordance with an act of the legislature. The members are Wm. K. Kavanaugh, of St. Louis, President of the Lakes to the Gulf Deep Waterway Association; Cyrus P. Wolbridge, President of the St. Louis Business Men's League; Ex-Gov. O. M. Dockey, of Gallatin; Walter S. Dickey, of Kansas City, head of a company which is to establish a barge line on the Missouri River, and Lawrence M. Jones, of Kansas City, President of a company which operated a boat line on the Missouri two years ago.

A number of changes are announced in connection with the separation of the Hocking Valley and Kanawha & Michigan Rys. from the Toledo & Ohio Central and Zanesville & Western Rys.:

Mr. Charles T. Lewis, formerly Vice-President of the Toledo & Ohio Central Ry., becomes President of that railway and of the Zanesville & Western Ry., succeeding Mr. N. Monsarrat, formerly President of all of the four railways named.

Mr. H. E. Speaks, formerly Superintendent of the Hocking Valley Ry., becomes General Superintendent of the Toledo & Ohio Central Ry.

Mr. M. S. Connors, formerly General Superintendent of the Zanesville & Western, Toledo & Ohio Central and Hocking Valley Rys., becomes General Superintendent of the Hocking Valley Ry.

Mr. T. M. Connors, formerly Superintendent of the Kanawha & Michigan Ry., becomes Superintendent of the Hocking Valley Ry., succeeding Mr. Speaks.

Obituary.

W. J. Mellor, Superintendent of Bridges and Buildings of the Louisiana & Texas R. R., died June 20 at his home in New Orleans. He had been employed on lines of the Southern Pacific system since 1883 and had been Superintendent of Bridges and Buildings for the past eleven years.

A. Bradshaw Holmes, Secretary and Treasurer of the Independent Pneumatic Tool Co. and the Aurora Automatic Machinery Co., of Chicago, Ill., died June 30, aged 31 years. His death was the result of injuries received in a fall from the piazza of his hotel. Mr. Holmes had been Secretary and Treasurer of the Independent Pneumatic Tool Co. since its organization. He was previously associated for a number of years with the Standard Pneumatic Tool Co. and the Rand Drill Co.

Albert P. Brayton, President and founder of the Pelton Water Wheel Co., died June 25 at his home in Oakland, Cal., at the age of 82 years. Mr. Brayton was born in Jefferson County, N. Y. He went to California in 1851 by way of Panama and was engaged for a few years in ranching and mining. In 1856 he became a member of the firm of Rankin, Brayton & Co., which established and operated the Pacific Iron Works in San Francisco. He left that company in 1888 and founded the Pelton Water Wheel Co.

Engineering Societies.

COMING MEETINGS.

OHIO ELECTRIC LIGHT ASSOCIATION.
July 13-15. Annual convention at Toledo, Ohio. Secy., D. L. Gaskill, Greenville, Ohio.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
July 15-16. Summer meeting at Indianapolis, Ind. Secy., W. M. Mackay, P. O. Box 1818, New York City.

NATIONAL IRRIGATION CONGRESS.
Aug. 9-14. Annual meeting at Spokane, Wash. Secy., B. A. Fowler, Phoenix, Ariz.

LEAGUE OF AMERICAN MUNICIPALITIES.
Aug. 25-27. Annual convention at Montreal, Can. Secy., John MacVicar, Des Moines, Iowa.

ASSOCIATION OF EDISON ILLUMINATING COMPANIES.
Aug. 31-Sept. 2. Annual meeting at Briarcliff Manor, N. Y. Secy., D. L. Huntington, Spokane, Wash.

TRAVELING ENGINEERS' ASSOCIATION.
Sept. 7-10. Annual convention at Denver, Colo. Secy., W. O. Thompson, N. Y. C. Car Shops, East Buffalo, N. Y.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 8-10. Annual convention at New York City. Secy., Willard Kent, 715 Tremont Temple, Boston, Mass.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Sept. 14-16. Annual convention at Atlantic City, N. J. Secy., Frank P. Foster, Corning, N. Y.

MICHIGAN GAS ASSOCIATION.
Sept. 14-17. Annual meeting on board steamer sailing from Detroit, Mich. Secy., A. P. Ewing, Gas Office Bldg., Detroit, Mich.

CUBAN SOCIETY OF ENGINEERS.—The society's headquarters have been recently established at Paseo de Marti 54, altos, Havana, where comfortable and attractive rooms have been secured. The secretary is Mr. Juan Manuel Planas.

INTERNATIONAL CONGRESS FOR MINING, METALLURGY, APPLIED MECHANICS AND PRACTICAL GEOLOGY.—The next session will be held at Düsseldorf, Rhenish Prussia, during the latter part of June, 1910. The meeting is expected to last about one week with one or two days devoted to each of the four sections given above in the society's name. The papers will take up the most important scientific and industrial questions of the day in the various fields covered and will be supplemented by excursions to places of geological interest.

NATIONAL ASSOCIATION OF CEMENT USERS.—After an investigation of the possibilities of holding next year's convention in New York City, with the exhibition of cement and concrete machinery and appliances at Madison Square Garden, the executive committee has decided that the financial obligations of such a move would be too burdensome and have accepted the invitation of the Cement Products Exhibition Co. to hold a joint exhibition and convention in Chicago next year. The annual meeting then will be held Feb. 17-24, 1910, in Chicago. Before this these two associations have held separate exhibitions, but the National Association has held a technical convention whereas the Chicago meeting was confined solely to an exhibition.

OHIO ELECTRIC LIGHT ASSOCIATION.—The following questions have been submitted to be answered at the 15th annual convention in Toledo, July 13-15:

1. What method should be pursued, in dealing with consumers, by a station changing its frequency from 125 cycles to 60 cycles?

2. In a town of 10,000 inhabitants, or smaller, with a small day load at the station, should the plant consist of three-phase generators with three-phase distribution, three-phase generators with single-phase distribution or single-phase generators with single-phase distribution?

3. In a station of 400-KW. capacity or less, what method should be followed in creating an interest in all the employees favorable to the station?

4. How should the third question be answered in the case of a large city station?

5. What is the best method of installing lightning arresters?

WESTERN SOCIETY OF ENGINEERS.—At the meeting held on June 16, at the society's rooms in Chicago, a paper on "The Reconstruction of Street Railway Tracks in Chicago" was read by Mr. George Weston, Chief Engineer of the Board of Supervising Engineers in charge of this work. The city ordinance requires the use of a grooved girder rail, and three types of track construction were developed by the above Board. In the first type the rails rest on steel tie-plates on steel I-beam ties (4 ft. apart) embedded in concrete, the ties having a wide top flange and narrower bottom flange. The rails are secured by channel-shaped clamps engaging the top of rail base and bottom of tie flange, and held in place by taper keys driven behind the clamps. In view of the rigidity of this track and the absence of cushioning to absorb shock and vibration this type of construction has not been used extensively.

The present standard construction has wooden ties 3 ft. apart, laid on concrete or on a rolled bed of broken stone, with concrete filled between and over the ties to carry the paving. The ties are treated by the zinc-chloride process, and are protected by steel tie-plates. Each rail is fastened to the tie by two screw spikes, and the tie-plate is attached to the tie (independently of these spikes) by two 2½-in. drive screws. The concrete base is used only where large stretches of track can be built, and the traffic temporarily diverted to other lines, so that the concrete may be given sufficient time to set (14 days being specified).

The rails are held to gage by flat tie-bars having threaded ends which pass through the rail webs. The gage is also maintained by the use of shoulder tie-plates, the shoulder fitting against the edge of the rail base. The rail joints are electrically welded, but the rails are connected to special work by bolted joints, a copper cable being laid through the special work from rail to rail. Manganese steel is used extensively for the special work.

Measurements made with a deflectometer indicated a maximum vertical movement of the rail of 5/64-in., this being at the rail joints. A large percentage of the tests showed no deflection. The movement at the joints appears to be due to the fact that a hole in the concrete is left at each joint, and is not filled until after the welding has been done. A horizontal movement of 1/64-in. was observed in a number of cases.